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September 1998

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Repair, Evaluation, Maintenance, and Rehabilitation Research Program

Innovative Methods for Levee Rehabilitation

by Edward B. Perry

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Prepared for Headquarters, U.S. Army Corps of Engineers

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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Innovative Methods for Levee Rehabilitation

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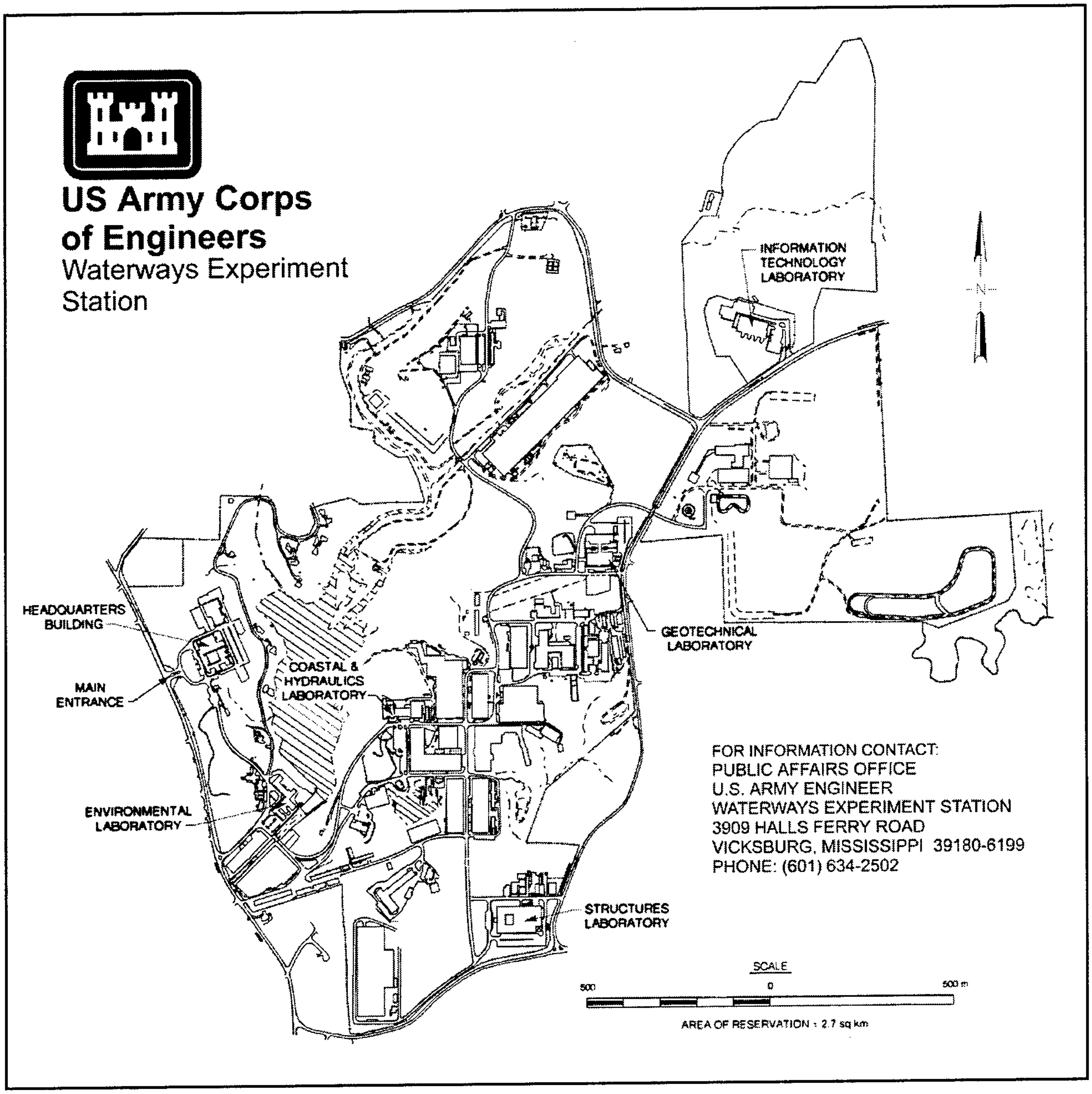

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Preface

The work described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Geotechnical Problem Area of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The work was performed under Civil Works Research Work Unit 32646, "Levee Rehabilitation." The REMR Technical Monitor was Mr. Arthur H. Walz (CECW-EG).

Mr. David Mathis (CERD-C) was the REMR Coordinator at the Directorate of Research and Development, HQUSACE. Mr. Harold Tohlen (CECW-O) and Dr. Tony C. Liu (CECW-EG) served as the REMR Overview Committee. The REMR Program Manager was Mr. William F. McCleese, U.S. Army Engineer Waterways Experiment Station (WES). Mr. Robert D. Bennett, Chief, Soil and Rock Mechanics Branch (S&RMB), Geotechnical Laboratory (GL), WES, was the Problem Area Leader.

The study was performed by Dr. Edward B. Perry, formerly Soil and Rock Mechanics Division, GL, under the general supervision of Mr. Bennett and Dr. William F. Marcuson III, Director, GL.

During publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Robin R. Cababa, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurements used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
gallons	3.785412	cubic decimeters
inches	2.54	centimeters
miles	1,609.347	meters
miles	1.609347	kilometers
pounds (force)	4.448222	newtons
pounds (force) per inch	175.12685	newtons per meter
pounds (force) per foot	14.593904	newtons per meter
pounds (force) per square inch	6.8947579	kilopascals
pounds (force) per square foot	0.04788	kilopascals
pounds (mass) per cubic foot	0.1570873	kilonewtons per cubic meter

1 Introduction

The U.S. Army Corps of Engineers (USACE) is responsible for 8,500 miles[1] of levees. Levees are subject to overtopping, current and wave attack on the riverside slope, surface erosion of slopes and crest resulting from rainfall, through-seepage causing softening and sloughing of the slope in the vicinity of the landside toe and associated piping problems, underseepage resulting in uplift pressures on the landside impervious top stratum with associated sand boils and piping problems, and slope instability in the form of deep-seated or shallow surface slides.

Table 1 shows conventional and innovative methods of levee rehabilitation for the various types of damage outlined above. This report provides details of innovative methods of levee rehabilitation. It does not cover operations during flood fighting, for example, using sandbags to raise a levee.

The innovative methods presented in this report are not intended to replace existing methods but rather to add to the repertoire of conventional methods available to the designer. For example, when levees are located in urban areas, the expense involved in obtaining necessary rights-of-way for conventional rehabilitation measures, such as slope flattening for slope instability, is prohibitive, leaving innovative methods as the only feasible solution.

Levees are not intended to serve as dams (water storage structures) but rather to hold back water for a short period of time as a flood crest passes through (normally for only a few days or weeks or a year). Recent floods, such as the Midwest flood of 1993, subjected levees to a period of high water which greatly exceeded those assumed in design of levees (Turk and Torrey unpublished). This resulted in conditions which exceeded those assumed in design. For example, the long periods of water retention produced saturation and softening and/or through- seepage. Wave wash occurred during periods of high winds over long fetches of floodwaters. In some cases overtopping occurred. None of these conditions are covered in design, and levees cannot be expected to survive these extreme conditions intact (Headquarters (HQUSACE) 1978).

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[1] A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page vi.

Table 1
Conventional and Innovative Rehabilitative Methods for Various Levee and Floodwall Problems

Problem	Rehabilitative Methods: Conventional	Rehabilitative Methods: Innovative
Overtopping	**Overtopping protection of levee crest and landside slope**	
	Rebuild	Cellular confinement system
	Vegetation	Reinforced grass
	Concrete slabs	Concrete block system
		Soil cement/roller compacted concrete
	Overtopping prevention (methods to raise the levee)	
	Steel sheetpiling	Cellular confinement system
	Earth capping (potato ridges)	Mechanically stabilized earth
	Sandbags	Inflatable structure
	Plywood flashboard with earth backing	Lightweight material (wood chips, tire chips, mechanically stabilized backfill, expanded polystyrene blocks
	Plywood mud box with earth fill	
	Precast post and panel wall	
Current and wave attack	Vegetation	Reinforced grass
	Revetment (riprap, concrete rubble, soil cement blocks, used auto tires, etc.)	Concrete block system
	Gabions	Soil cement/roller compacted concrete
Surface erosion due to rainfall	Vegetation	Turf reinforcement mats
	Chemical stabilization	
Through-seepage	Toe drain	Bio-polymer chimney drain
	Conventional chimney drain	
Underseepage	Conventional toe trench	Bio-polymer toe trench
	Conventional cutoffs	Jet grouted cutoff
	Riverside blanket	
	Landside seepage berm	
	Pressure relief wells	
Slope instability	Drainage	Reinforced soil slope
	Remove and replace soil (slope flattening and benching)	Soil nailing
	Conventional restraint structure	Pin piles
	Chemical treatment by mixing in place (cement, lime, fly ash, etc.)	Stone-fill trenches
		Randomly distributed synthetic fibers
		Restraint structure
		Geosynthetic drainage system
		Lime-fly ash injection
		Anchored geosynthetic system

2 Overtopping

Background

Overtopping may occur during periods of flood due to insufficient freeboard. Local overtopping may occur due to low spots along the levee resulting from settlement of the levee, slope instability, etc. High water can also aggravate conditions, such as through-seepage and slope instability, and can combine with overtopping to cause failure of the levee. Survival of the levee during overtopping depends on the duration and flow conditions during overtopping (depth of flow, hydraulic shear stress exerted on the soil, etc.) and whether or not the levee provides sufficient protection against the overtopping flow. As mentioned previously, normally levees are not designed to withstand overtopping. However, since overtopping has occurred during recent floods, this report provides information on protection against overtopping.

A discussion of the mechanisms involved in overtopping is given by Gilbert and Miller (1991) and Dodge (1988). Suggested design assumptions and procedures to use when considering the potential flood overtopping of levees and floodwalls are given by Smith and Munsey (1986). Survival during overtopping is closely related to conditions at the landside toe of the levee. Levees constructed of cohesive soil show first signs of erosion distress at the toe as a result of energy dissipation of overflowing water. Removal of toe material causes undercutting and progressive removal of the landside slope of the levee. Reinforcement or protection of the downstream toe area delays destruction of the embankment (Miller 1990). The maximum flow velocity during overtopping depends on the height of the levee, difference in elevation between river stage and tailwater level, landside slope angle, and discharge rate and hydraulic roughness of the landside slope.

Two courses of action are possible to address overtopping. Overtopping protection can be provided to the crest and landside slope of the levee, or the levee can be raised to prevent overtopping.

Overtopping Protection of Levee Crest and Downstream Slope

As shown previously in Table 1, conventional overtopping protection methods include rebuilding, vegetation, and concrete slabs. Innovative overtopping protection methods, developed primarily for dams, include cellular confinement systems, reinforced grass, concrete block systems, and soil cement/ roller compacted concrete (RCC) (American Society of Civil Engineers (ASCE) 1994b, Powledge et al. 1989a,b).

Cellular confinement system

During the late 1970's, the sand grid or cellular confinement system was developed at the U.S. Army Engineer Waterways Experiment Station (USAEWES) in a cooperative research effort with Presto Products Company, Appleton, WI (Figure 1). The original concept involved confining and compacting sand in grid elements to use in expedient road construction, airfield damage repair, field fortifications, slope erosion control, etc. (Purinton and Harrison 1994). Early experiments showed that cells filled with stone had relatively low resistance to flowing water (Chen and Anderson 1987, Clopper and Chen 1988, Hughes 1994). Movement of 2-in.-diam stone (approximately the largest size used) in the cellular confinement system occurred when the flow

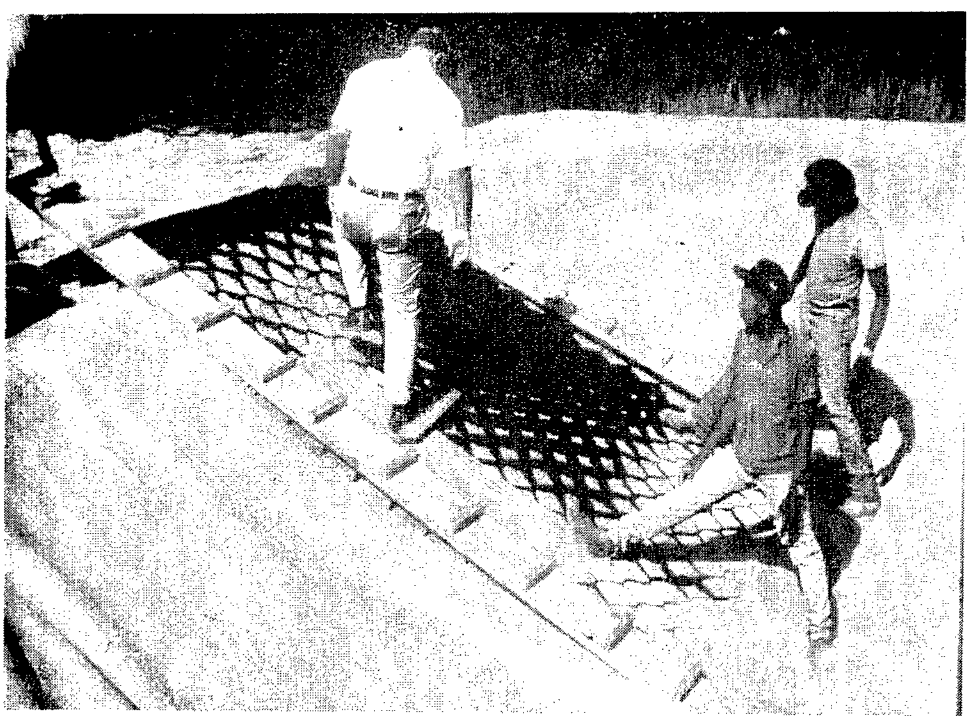


Figure 1. Geocells installed on slope to prevent erosion and foster vegetation establishment

velocity exceeded about 6 ft/sec. For the test conditions, a flow velocity of 6 ft/sec is equivalent to a hydraulic shear stress of 0.7 lb/sq ft (Chen and Anderson 1987). Therefore, concrete-filled cells are recommended when cellular confinement systems are used as overtopping protection for levees and dams (Presto Products Company 1992; Snef 1995; Crowe, Snef, and Martin 1995b). Such systems may require a geotextile, a drainage composite or stone base, anchors (helical and/or duckbill depending on the soil conditions) into the levee, and/or weep holes to allow water to escape from behind the concrete (Figure 2). Perforated cells are available to provide for drainage through the cells (Martin, Snef, and Crowe 1998). Cellular confinement systems may also be stacked vertically to raise a levee to prevent overtopping (discussed below).

Reinforced grass

Grass has long been used to protect earth structures from rainfall erosion, flow of water in channels, and intermittent flow due to embankment overtopping. Figure 3 shows the limiting velocity versus time flow duration for plain grass (Hewlett, Boorman, and Bramley 1987). Vegetation on the levee crest and downstream slope will provide surface resistance, reduce the speed of flow, and strengthen the soil by its root system (Coppin and Richards 1990; Temple et al. 1987; Gray et al. 1991; Hughes and Hoskins 1994; Barker 1995). Guidelines for landscape planting on levees and floodwalls is given in EM 1110-2-301 (HQUSACE 1993a).

Reinforced grass refers to a grass surface that has been artificially augmented with an open structural covering or armor layer (geotextile, geogrid, concrete blocks, etc.) to increase its resistance to erosion above that of the grass alone as shown in Figures 3 to 5 (Hewlett, Boorman, and Bramley 1987; Hoffman 1990). Field investigations using reinforced grass indicates that geotextile systems tend to fail when flow between the topsoil and fabric produces uplift pressure at the interface leading to exposure and subsequent erosion of the underlying soil (Powledge et al. 1989a,b). Failure in field tests have occurred due to poor anchorage or stretching of the underlying geotextile (Frizell et al. 1991). Grass roots provide interlocking and increased resistance to lift off and local shear present during overtopping flow (Gray and Sotir 1996; Schiechtl and Stern 1996, 1997). As shown in Figure 3, for a flow duration of 2 days (significance of which is discussed below), a geotextile reinforced grass mat may uplift when flow velocities exceed about 12.5 ft/sec.

Reinforced grass is not recommended for use if the flow duration is longer than about 2 days (Hewlett, Boorman, and Bramley 1987). Submergence and root waterlogging will begin to kill the grass after a few days of overtopping (Whitlow and Harris 1979, Gray and Sotir 1996).[1] Recent floods on the Upper

[1] This time period could possibly be extended by using more flood-resistant grass and/or providing additional anchorage (helical and/or duckbill, depending on the soil conditions) to the reinforced grass system.

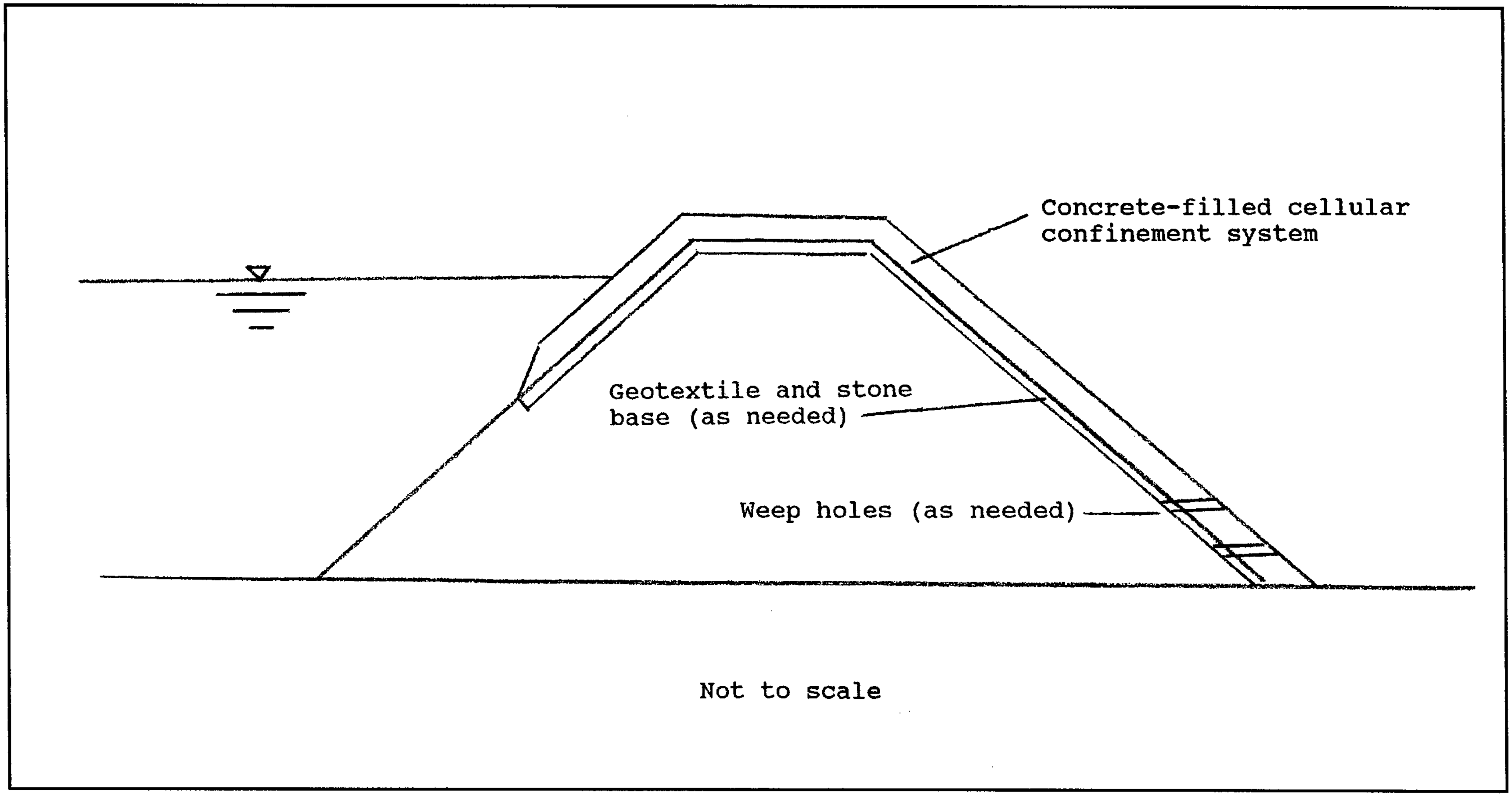
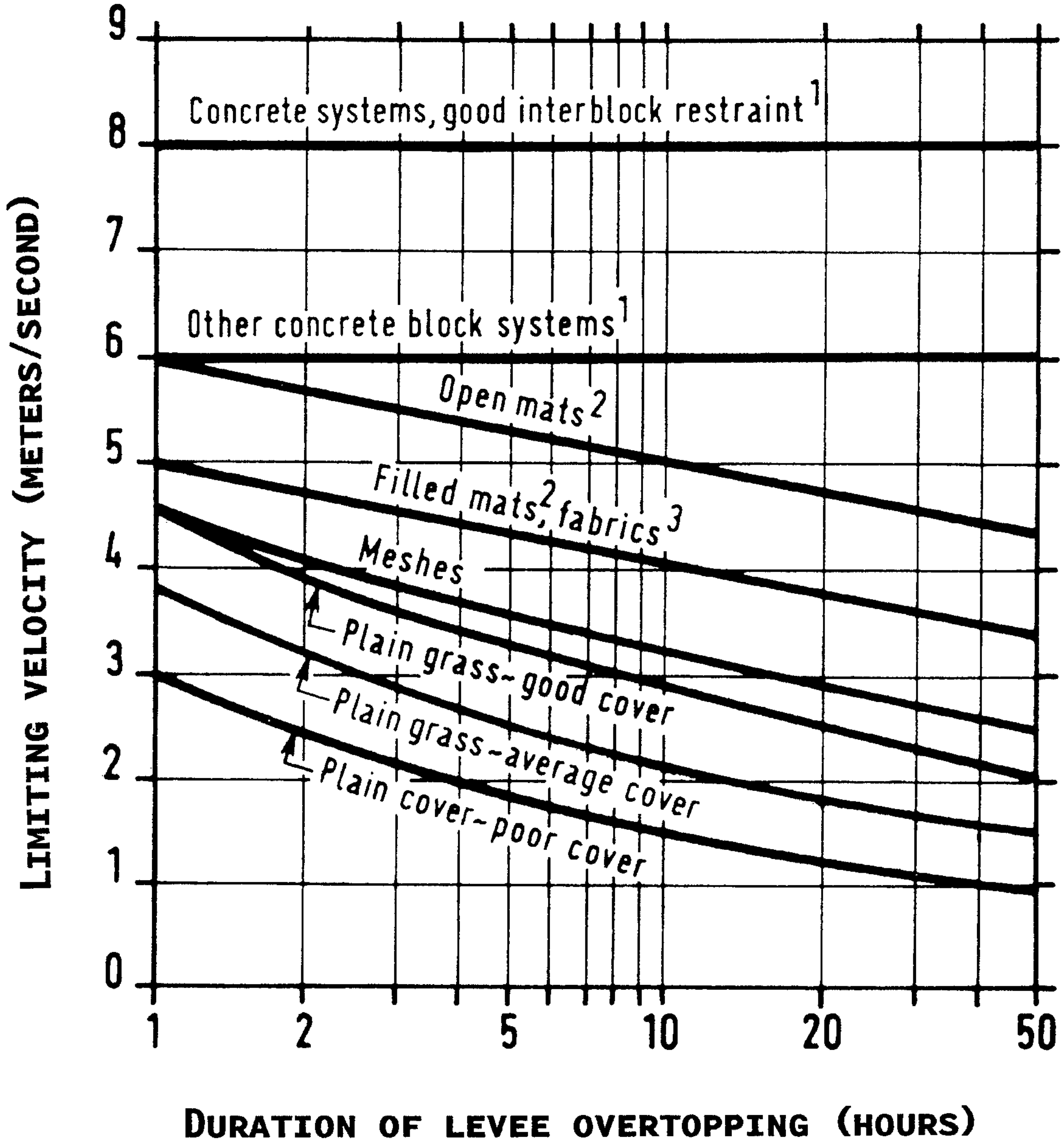


Figure 2. Concrete-filled cellular confinement system for overtopping protection on levee



Notes:
1. Minimum superficial mass 135 kg/m^2, see Section 4.3.3 for other criteria.
2. Minimum nominal thickness 20 mm.
3. Installed within 20 mm of soil surface, or in conjunction with a surface mesh.
4. See Section 4.3.2 for other criteria for geotextile reinforcement.
5. These graphs should only be used for erosion resistance to unidirectional flow. Values are based on available experience and information at date of this report.
6. All reinforced grass values assume well-established, good grass cover.
7. Other criteria (such as short-term protection, ease of installation and management, susceptibility to vandalism, etc) must be considered in choice of reinforcement.

Figure 3. Limiting velocities for erosion resistance of plain and reinforced grass versus duration of flow (courtesy of Hewlett, Boorman, and Bramley 1987)

Figure 4. Geosynthetic erosion control matting for reinforced grass

Mississippi River indicated flow durations of about 1 week would have occurred if overtopped levees had held (Turk and Torrey unpublished).

Reinforced grass as compared to nonreinforced grass has economic and environmental advantages over conventional engineering materials such as concrete and rock. For example, the development of weak spots by concentrated traffic, livestock damage, or drought will be retarded by the reinforcement. Also, reinforcement provides lateral continuity between grass plants and reduces the risk of localized failure due to erosion, shallow slippage, or "rolling up" of the soil/grass mat.

Disadvantages include root survival during drought and requirements for management of the vegetation. Although reinforced grass is generally more economical than conventional engineering materials in capital cost, it can be more expensive to maintain. Also, reinforced grass may be susceptible to damage during grass establishment prior to development of the root system (Hewlett, Boorman, and Bramley 1987).

The design of a reinforced grass system for protection against levee overtopping involves hydraulic, geotechnical, and botanical considerations. Anchorage of the reinforced grass system, using helical and/or duckbill anchors, may be required to prevent uplift, local erosion, vulnerability to vandalism, etc. Details such as curving the downstream edge of the levee to give a smooth flow

Figure 5. Installation of geosynthetic erosion control matting for reinforced grass

profile, energy dissipation at the downstream toe of the levee, preventing buildup of pressures below the open structural covering or armor layer, and root survival during drought conditions must be considered. If tailwater is present at the toe of the levee, fluctuating pressures associated with a hydraulic jump must be addressed. Particular attention must be paid to edge details, i.e., how the composite system is terminated at the crest, toe, and sides as well as laps and joints. The armor layer must be relatively permeable to inhibit buildup of pressures at the subsoil/armor interface (Hewlett, Boorman, and Bramley 1987).

In summary, reinforced grass is a viable option for overtopping protection of the levee crest and downstream slope when the duration of flow during flood is less than 2 days and poor cover of newly sown grass during the first growing season is acceptable. Particular attention must be paid to design details such as anchorage of the reinforced grass system, geometry of the downstream edge of the levee, flow conditions at the downstream toe of the levee, fluctuating pressures associated with a hydraulic jump (if tailwater is present at the toe of the levee), edge details, and root survival during drought.

Concrete block system

A concrete block system is a matrix of individual concrete blocks assembled to form a large mat. Blocks are 4 to 9 in. thick and 1 to 2 ft square in plan with openings penetrating the entire block as shown in Figure 6. Blocks are usually designed to be intermeshing or interlocking and many units are patented. The blocks may be hand-placed (articulated) or threaded with polyester or galvanized steel cable to form prefabricated mattresses placed with a spreader-bar and large crane (Koutsourals 1994; Wooten, Powledge, and Whiteside 1992). Blocks may be solid or have open cells to permit uplift pressure relief and vegetation growth (ASCE 1994b). Vegetated systems provide environmental attractiveness (aesthetic and habitat-enhancing) and a small improvement in uplift resistance when the root system is developed (Frizell 1991). Depending on the subsoil conditions, a geotextile may be required between the concrete blocks and the soil.

A stable foundation is required for placement of the concrete blocks. For placement on the crest and downstream slope of the levee, the surface should be stripped to expose the soil and compacted as required to prevent settlement of the subsoil which could cause displacement of the concrete block units and ultimate failure of the system. Blocks should be placed from the toe up the slope to avoid putting the interlocking units in tension (Fuller 1992).

As stated previously, field investigations using of reinforced grass indicates that geotextile systems tend to fail when flow between the topsoil and fabric produces uplift pressure at the interface leading to exposure and subsequent erosion of the underlying soil (Powledge et al. 1989a,b). For concrete block systems, failure is triggered when there is loss of “intimate contact” between a block or group of blocks and the underlying soil. For blocks which are not cabled together, this occurs when flow-induced pressure fluctuations or uplift

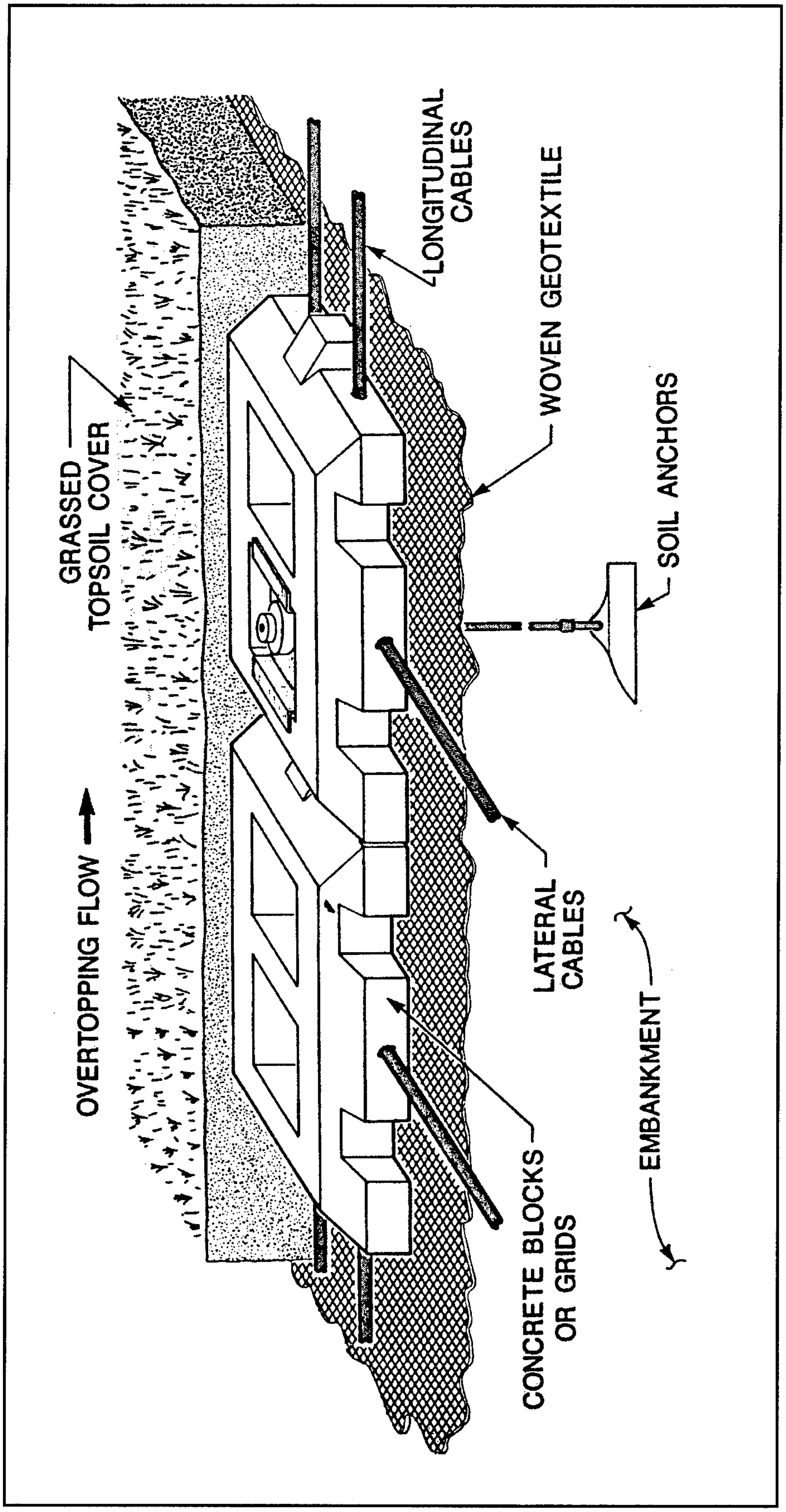


Figure 6. Schematic cutaway of articulated concrete block system

pressures exceed the interblock forces and the block begins to lift. The exposed edge of the lifted block experiences increased lift and form drag allowing the block to be dragged out leading to progressive failure of the remaining blocks (Hewlett, Boorman, and Bramley 1987; Clopper and Chen 1988). Concrete block systems have performed well up to flow velocities of 25 ft/sec, which is the limit for present test facilities (Frizell et al. 1991; Clopper 1989; Powledge, Rhone, and Clopper 1991). For the test conditions, a flow velocity of 25 ft/sec is equivalent to a hydraulic shear stress of 15 lb/sq ft (Clopper and Chen 1988).

Anchorage of the concrete block system, using helical and/or duckbill anchors, may be required to prevent uplift, local erosion, vulnerability to vandalism, etc. Since the primary resistance of the blocks to movement is due to the interlocking forces between individual blocks, weight of the blocks, and anchors installed, the hydraulic stability of the system is independent of flow duration (Hewlett, Boorman, and Bramley 1987; Koutsourals 1994). The small anchorage provided by the grass roots, which would take time to develop following seeding and would deteriorate with duration of flow during flood, is negligible and not considered in design.

Advantages of concrete block systems include the ability to sustain relativity high flow velocities (in excess of 25 ft/sec) immediately following installation, hydraulic stability of the system is independent of flow duration, ability of blocks with open cells to release excess hydrostatic pressures. and ability to accommodate small subgrade movements caused by settlement, frost heave, etc. (Koutsourals 1994). The disadvantages of concrete block systems are that the interlocking feature between units must be maintained. Routine maintenance is also required to prevent bushes from growing through the openings. For systems which are not cabled together, if one block is lost, other units soon dislodge and complete failure can result. Also, most concrete block systems have relatively smooth faces which could lead to significantly higher wave runup (compared to dumped rock). Wave runup is the vertical height above the stillwater level to which the uprush from a wave will rise on the levee. It is not the distance measured along the inclined surface.

In summary, for overtopping protection of the levee crest and landside slope, a concrete block system with interlocking open cell vegetated units would be appropriate. An underlying geotextile and anchor system might be required depending on subsoil conditions. A design procedure for concrete block systems is given by Clopper (1990, 1991). Computer programs for the design of channels and slope protection are available from companies marketing concrete block systems (American Excelsior Company 1996, ARMORTEC 1997, and Synthetic Industries 1995). Energy dissipation at the downstream toe of the concrete block system should be addressed in the design. A concrete block system should be well maintained, and any damaged or missing blocks should be replaced (Powledge and Pravdivets 1992; Powledge, Wooten, and Whiteside 1991).

Soil cement/roller-compacted concrete

Soil cement is formed by blending and compacting a mixture of coarse sand or gravelly soils and portland cement. RCC differs from soil cement in that it contains coarse (greater than 3/4-in.-diam) aggregate and develops hardened properties similar to those of conventionally placed concrete. The principal difference between RCC and conventional concrete is that RCC has an aggregate gradation and paste content suitable for compaction by a vibratory roller (McDonald and Curtis 1997). Also, conventional concrete has a formed or screeded surface free of surface imperfections that cause cavitation erosion at high (40 ft/sec) velocities (HQUSACE 1993b, ASCE 1994a). The major advantages of soil cement and/or RCC are cost saving and speed of construction.

Both soil cement and RCC have been used in dam construction and/or modification and have potential application for use in protection against overtopping of levees. Soil cement has been used extensively by the U.S. Bureau of Reclamation (USBR) for upstream slope protection of earthfill dams where suitable riprap was not available near the site (DeGroot 1971, USBR 1987). RCC has been used to build dams and in dam modification (Hansen 1985, Hansen and Guice 1988, Hansen and McLean 1992). The U.S. Army Engineer District (USAED), Portland, completed Willow Creek Dam, the first RCC dam in the United States, in 1982 (HQUSACE 1992). RCC was first used as overtopping protection on Addicks and Barker Dams by the USAED, Galveston, in 1988. RCC has become the most widely used method for spillway modification and overtopping protection and has been used on 50 embankment dams since 1988 (Hansen and Reinhardt 1991; Hansen 1992, 1993; McLean and Hansen 1993; Hansen 1996).

Soil cement or RCC may be placed and compacted in stairstep horizontal layers (Figure 7) or by plating a single layer placed parallel to the slope. While the plating method uses less material and is more economical, it is difficult to install on steep (20 percent or greater) slopes and the smooth face results in greater wave runup (Bingham, Schweiger, and Holderbaum 1992; Portland Cement Association (PCA) 1992). When the plating method is used, it should be keyed in on the riverside to prevent headcutting, at the landside to prevent slippage down the slope (failure to tie in the upstream end of the RCC overflow protection at Addicks and Barker Dams contributed to a longitudinal separation crack between the crest and upstream slope (McDonald and Curtis 1997)), and at the terminus ends to prevent flanking of the structure. Watertight expansion joints, at appropriate spacing, should be considered in the RCC section. When the stairstep method is used, the width of the RCC protection is usually controlled by the construction equipment (8 ft is common). This produces a minimum thickness of RCC measured perpendicular to the slope of 2 to 3 ft, depending upon the existing slope of the levee. The stairstep method provides energy dissipation due to the stepped configuration on the downstream slope (ASCE 1994b).

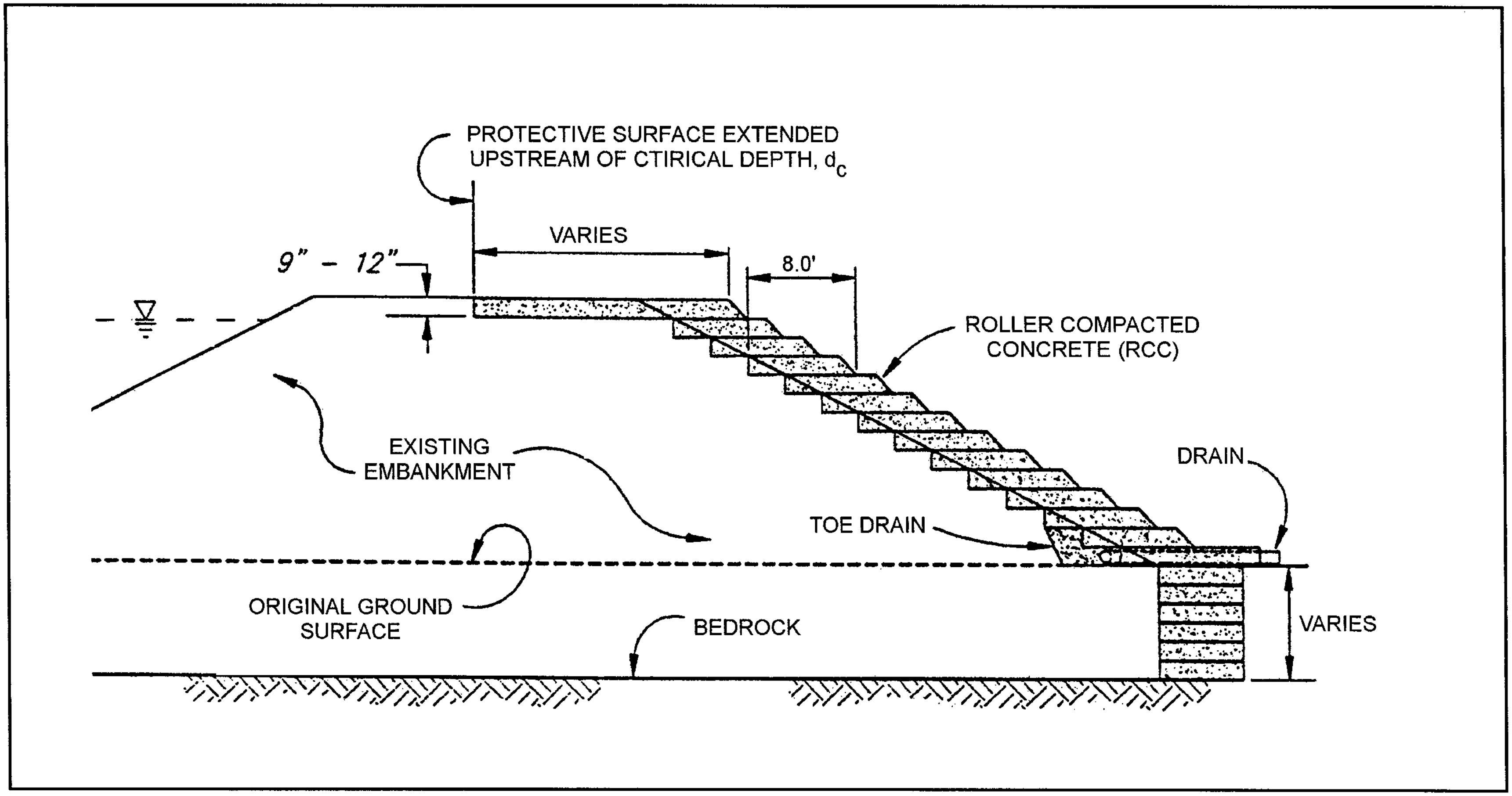


Figure 7. Typical modification of dam using RCC (courtesy of Powledge and Pravdivets 1992)

A drain should be placed behind the soil cement and/or RCC to provide for dissipation of excess pore water pressures in the levee. If the levee is relatively free draining, a toe drain with pipes exiting through the soil cement or RCC above the anticipated tailwater elevation will suffice. For more impermeable levee materials, a sand-gravel drainage layer should be placed directly beneath the soil cement or RCC. For critical locations, a geotextile and/or gravel drainage layers may be required to reduce uplift pressures beneath the soil cement or RCC.

Details concerning the design and construction of soil cement are available (PCA 1992). The results of wet-dry, freeze-thaw, and weight-loss criteria determine the cement content required. This cement content is increased by 2 percent for erosion resistance in a hydraulic application giving a total cement content of 10 to 12 percent by compacted volume of soil cement for typical environmental conditions and soils. The design and construction of RCC are covered in EM 1110-2-2006, ETL 1110-2-343, EP 1110-2-12 (seismic design) (HQUSACE 1992, 1993b, 1995c; USBR 1985; Morrison-Knudsen Engineers, Inc. 1986).

Erosion of soil cement and/or RCC used in overflow protection of levees can occur due to the hydraulic shear stress exerted by the flowing water; abrasive action from sand, gravel, or other waterborne debris; and cavitation from surface imperfections at flow velocities as low as 40 ft/sec (HQUSACE 1993b, ASCE 1994a). Overtopping flow over a levee would normally not be expected to carry coarse sediment which would abrade the soil cement or RCC protection. If coarse sediment was present in the overtopping flow, soil cement would not be used. If coarse sediment was present in the overtopping flow, an RCC mixture with a low water-cement ratio and larger-size aggregates could be used to provide erosion resistance equal to conventional concrete with similar ingredients (McLean and Hansen 1993). The method of construction employed for soil cement and RCC results in surface imperfections. Therefore, cavitation erosion would be expected if flow velocities exceed 40 ft/sec.

The upper limit of flow velocity (below 40 ft/sec where cavitation erosion occurs) above which appreciable erosion occurs for soil cement and RCC has not been established. Erosion experiments conducted on soil cement in the laboratory (Akky 1974) and in the field (Clopper and Chen 1988) indicate for clear water flow (no abrasive action) over a smooth surface (no cavitation from surface imperfections) little if any erosion occurs for flow velocities up to 20 ft/sec (for field experiments, limitations were imposed by embankment slope and depth of flow of water) and for flow durations of up to 10 hr. For the test conditions, a flow velocity of 20 ft/sec is equivalent to a hydraulic shear stress of 45 lb/sq ft (Clopper and Chen 1988). Since soil cement has been used in hydraulic applications with appreciable velocities and flow durations of several weeks or months, it is unlikely that significant erosion would occur during the lifetime of the levee-overtopping protection. Erosion experiments conducted on RCC in the laboratory (Saucier 1984) indicate for clear water flow (no abrasive action) little if any erosion occurred for flow velocities up to 35 ft/sec (limitations imposed by equipment) and for flow durations of up to 20 hr. For the same

reasons stated for soil cement, it is believed that flow duration would not be a factor in the use of RCC in levee protection. Also, as stated previously, RCC with increased strength and larger size aggregates could be used for coarse sediment present in the overtopping flow (McLean and Hansen 1993).

In summary, for nonsediment laden flows, soil cement could provide protection for flow velocities up to 20 ft/sec (upper limit of available test data). RCC could be used for flows with or without coarse sediment present and flow velocities up to 35 ft/sec (upper limit of available experimental data). In the rare case where coarse sediment was present in the overtopping flow which would abrade the RCC protection, RCC with increased strength and larger size aggregates could be used (McLean and Hansen 1993). If flow velocities exceeded 40 ft/sec, surface imperfections which would be present in the RCC would result in cavitation erosion. If the cumulative amount of the cavitation erosion which occurred during the intermittent periods of overtopping flow was unacceptable, a conventional concrete topping or facing (with a formed or screeded smooth surface) could be placed over the RCC to prevent cavitation erosion (HQUSACE 1993b; ASCE 1994a).

Overview of innovative overtopping protection methods

A brief overview of various innovative overtopping protection methods applicable to levees is given in Table 2 (Campbell 1993, Campbell and Harrison 1995). As shown in Table 2, a cellular confinement system using concrete-filled cells would be applicable for use when rapid, rather expensive, construction was justified and subsequent raising of the levee was anticipated. Reinforced grass would have limited application, because it would take several months (or years) for the root system to develop and it would be limited (unless more flood-resistant grass and/or additional anchorage was provided) to flow durations less than 2 days. Concrete block systems would be applicable for flow velocities up to 25 ft/sec with hydraulic stability independent of flow duration. Soil cement would be cost competitive with concrete block systems for nonsediment laden flows with velocities up to 20 ft/sec (upper limit of available experimental data). RCC would cost slightly more than soil cement and would be applicable for nonsediment laden flows with velocities up to 35 ft/sec (upper limit of available experimental data). If coarse sediment was present in the overtopping flow, RCC with increased strength and larger size aggregates could be used (McLean and Hansen 1993). If flow velocities exceeded 40 ft/sec and the cumulative amount of cavitation erosion which occurred during periods of levee overtopping was unacceptable, a conventional concrete topping or facing (with a formed or screeded surface) could be constructed over the RCC to extend the range of acceptable flow velocity (HQUSACE 1993b; ASCE 1994a).

Table 2
Overview of Innovative Rehabilitative Methods for Overtopping Protection of Levee Crest and Landside Slope

Rehabilitative Method	Applicable Conditions	Advantages	Disadvantages
Cellular confinement system	Cells must be concrete-filled	Easy to transport and construct	High cost of concrete fill
		May be stacked vertically to raise levee	Must provide drainage to relieve excess hydrostatic pressure
Reinforced grass	Flow velocity depends on duration (Figure 3)	Low capital costs	Susceptible to damage prior to development of root system
	Flow duration less than 2 days[1]	Reduces risk of localized failure	More expensive to maintain
			Root system may not survive drought
Concrete block system	Flow velocities up to 25 ft/sec (hydraulic shear stress of 15 lb/sq ft	Hydraulic stability is independent of flow duration	If blocks are not cabled together, system could fail if one block is lost
			Smooth face gives higher wave runup
Soil cement	Flow velocities up to 20 ft/sec or hydraulic shear stress of 45 lb/sq ft (upper limit of available experimental data)	Relatively low cost	Not applicable if coarse sediment present in flow
	Nonsediment laden flow	Speed of construction	Must provide drainage to relieve excess hydrostatic pressure
		Requires no coarse aggregate for mix	
RCC	Flow velocities up to 35 ft/sec (upper limit of available experimental data)	Relatively low cost	Relatively rough surface susceptible to cavitation erosion if flow velocities exceed 40 ft/sec
	Nonsediment laden flow[2]	Speed of construction	Must provide drainage to relieve excess hydrostatic pressure
	Flow velocities over 40 ft/sec[3]	High strength and erosion resistance	

[1] This time period could possibly be extended by using more flood-resistant grass and/or providing additional anchorage (helical and/or duckbill depending on the soil conditions) to the reinforced grass system.
[2] If coarse sediment was present in the overtopping flow, RCC with increased strength and larger size aggregates could be used (McLean and Hansen 1993).
[3] If flow velocities exceeded 40 ft/sec and the cumulative amount of cavitation erosion which occurred during periods of levee overtopping was unacceptable, a conventional concrete topping or facing (with a formed or screeded surface) could be constructed over the RCC to extend the range of acceptable flow velocity (HQUSACE 1993b; ASCE 1994a).

Overtopping Prevention (Methods to Raise the Levee)

As shown previously in Table 1, conventional overtopping prevention (methods to raise the levee) includes: driving steel sheetpiling, earth capping (potato ridges), sandbags, plywood flashboard with earth backing, plywood mud box with earth fill, and precast post and panel wall (Markle and Taylor 1988). Innovative overtopping prevention (methods to raise the levee) includes cellular confinement systems, mechanically stabilized earth, inflatable structure, and lightweight material (wood chips, tire chips, and expanded polystyrene blocks). While these innovative overtopping protection methods have had limited application to levees, there has been some application to dams ASCE 1994b; Powledge et al. 1989a,b).

Cellular confinement system

During the early 1980's, WES conducted preliminary experiments of a cellular confinement system (plastic grids filled with masonry sand) stacked vertically and subjected to static water level and cycles of wave attack. The results indicated the feasibility of using such a system to raise a levee to prevent overtopping (Markle and Taylor 1988). This concept was subsequently refined (Figure 8) by using a notched plastic grid for horizontal stability, placing a sand bentonite mixture in the interior cells to prevent through seepage and provide wave protection, if needed, filling the outer cells on the riverside with concrete (Snef 1995; Crowe, Snef, and Martin 1995a,b). The construction of vertical walls using a cellular confinement system is detailed by Torrey and Davidson (1995).

Mechanically stabilized earth (MSE)

Within the past 30 years, a number of wall systems utilizing MSE, as well as new types of gravity walls, have been developed (EM 1110-2-2502 (HQUSACE 1989)). Typical applications of MSE systems include construction of retaining walls and abutment structures, retention of excavations, and repair of slope failures. Walls are defined as structures with face inclinations of 70 to 90 deg and slopes as structures with face inclinations of less than 70 deg (Berg 1993). There has been at least one application of MSE construction to raise a dam to prevent overtopping. As shown in Figure 9, a 20-ft-high, 900-ft-long, reinforced earth retaining wall was used by the USBR in 1982 to raise Lake Sherburne Dam (Duster 1984, Engemoen 1993).

MSE systems have three major components: reinforcements, backfill, and facing elements. While most MSE systems use galvanized steel (aluminum alloys and stainless steels did not work), geogrid polymeric reinforcement, which gives greater economy (Figure 10), is used with increasing frequency (Holtz, Christopher, and Berg 1995; Tatsuoka and Leshchinsky 1994; Elias 1997). The

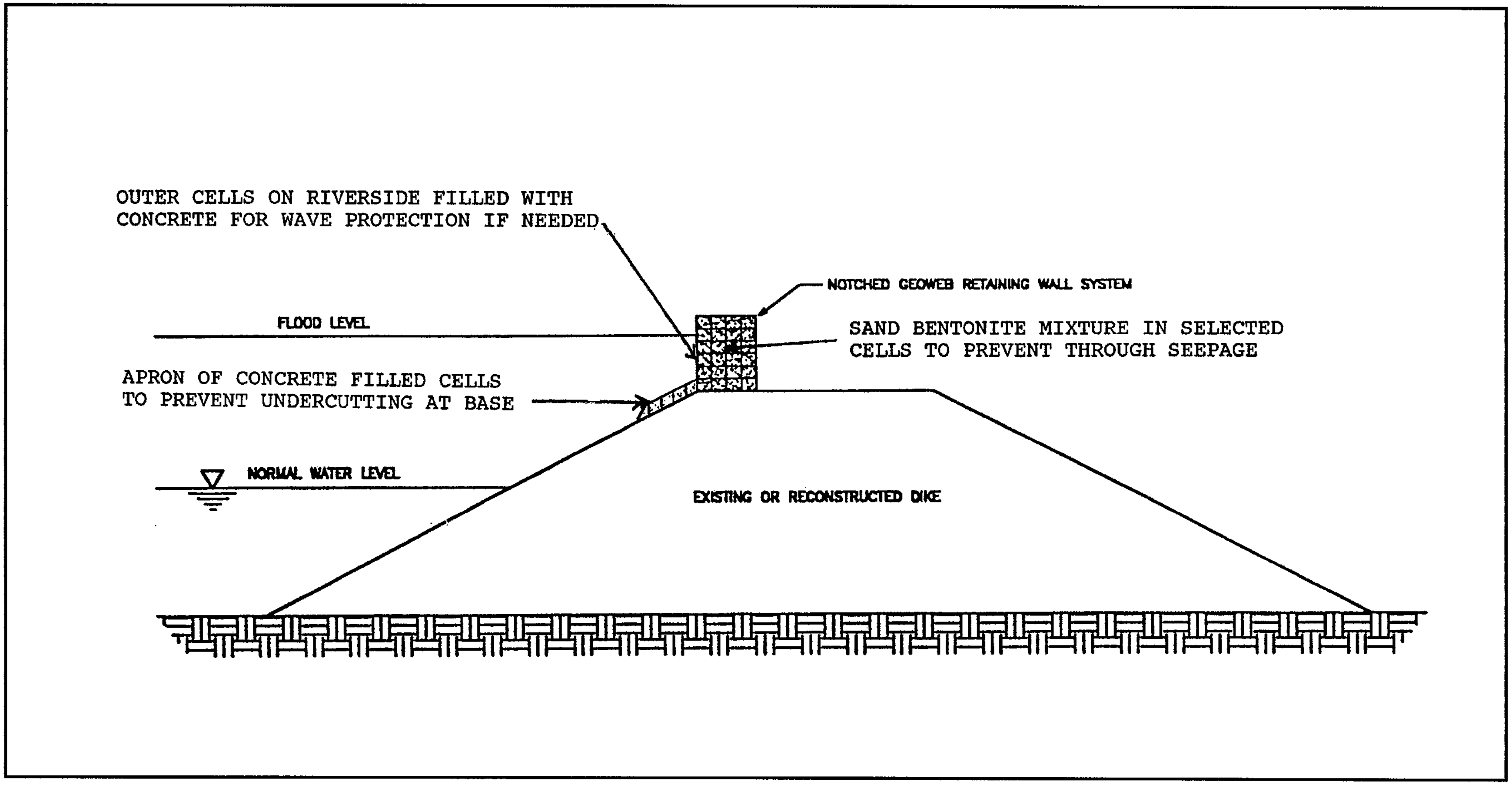


Figure 8. Cellular confinement system to raise a levee to prevent overtopping (courtesy of Snef 1995)

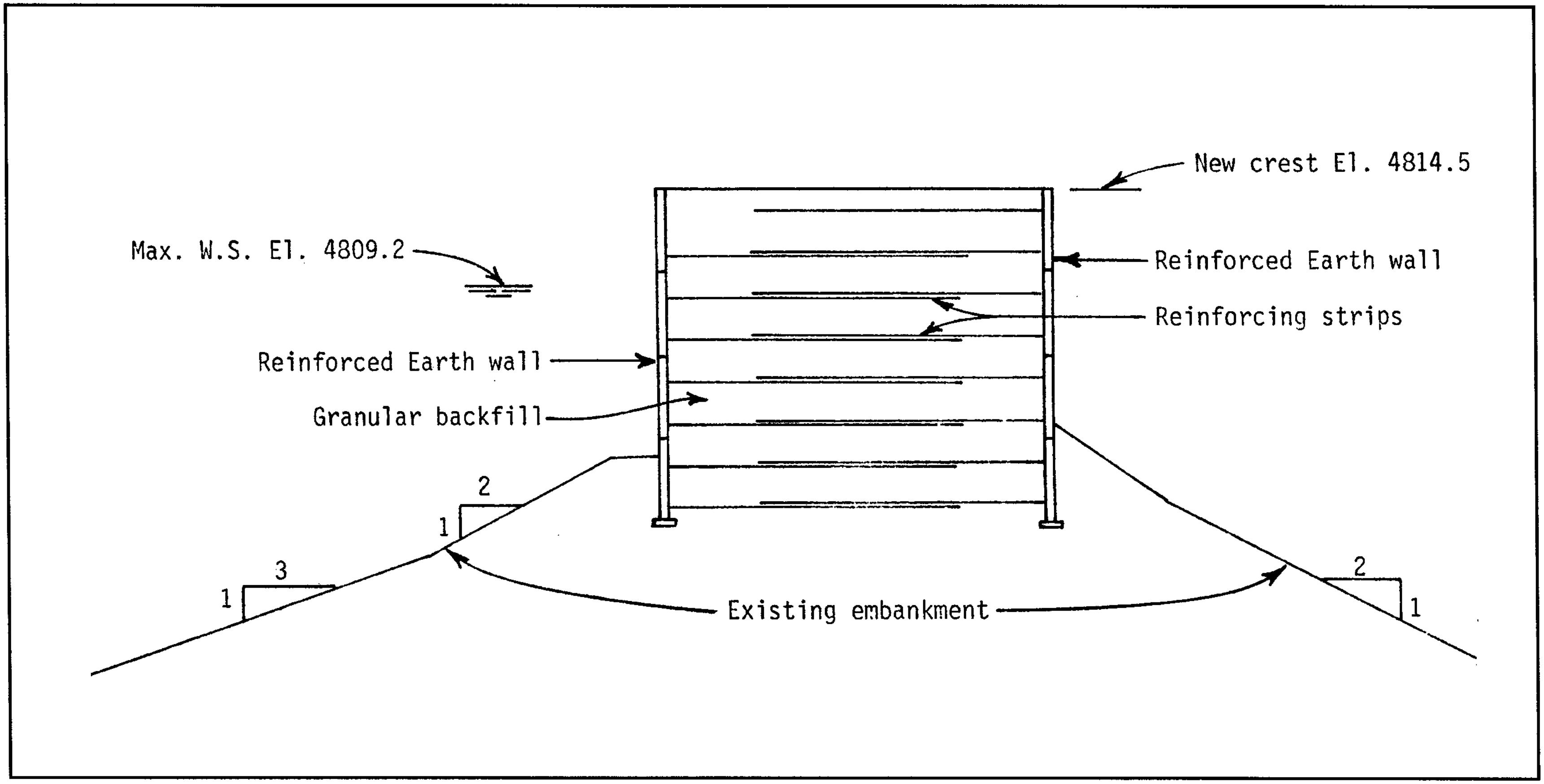


Figure 9. Reinforced earth retaining wall system used by the U.S. Bureau of Reclamation to raise Lake Sherburne Dam (courtesy of Duster 1984)

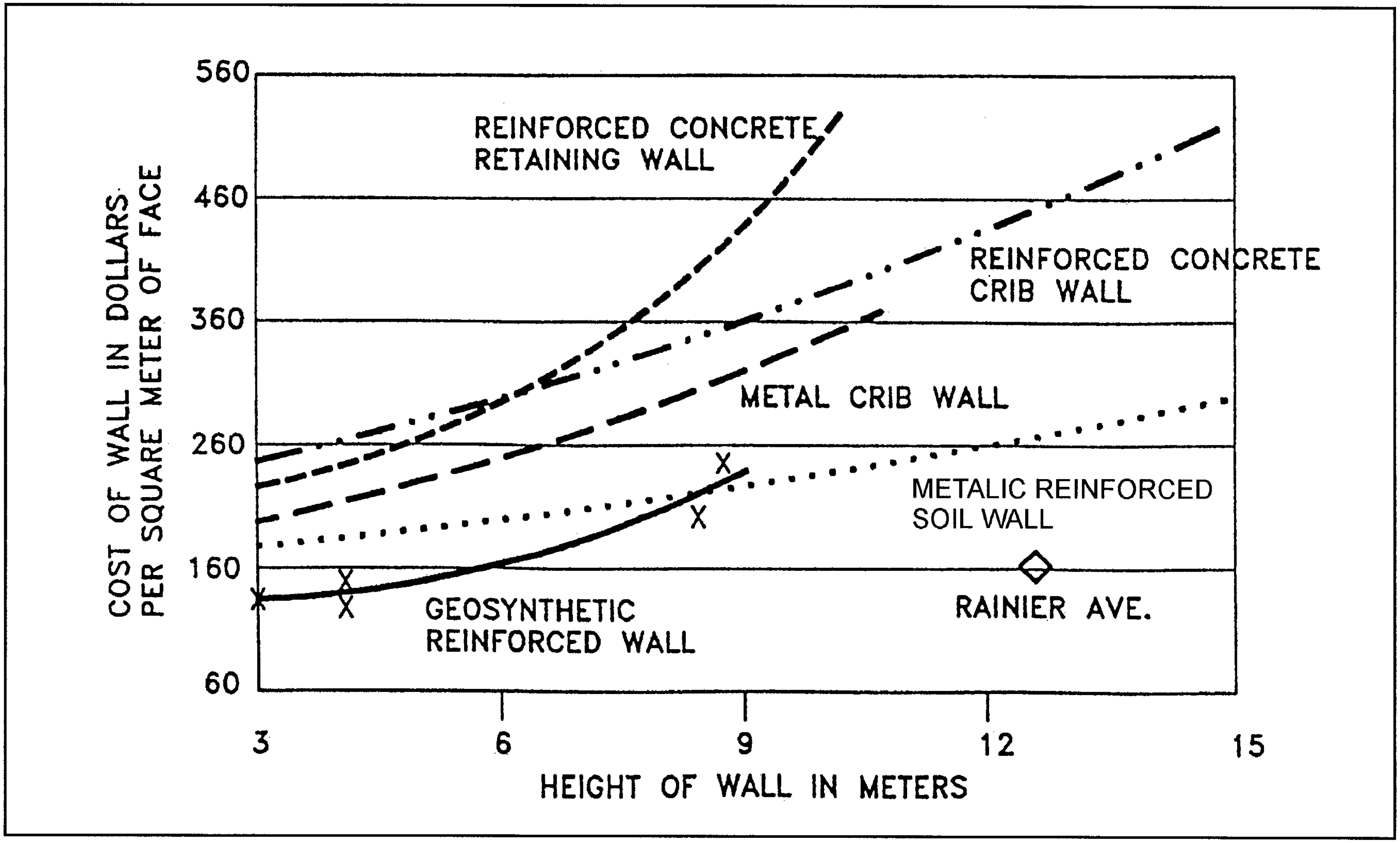


Figure 10. Cost comparison of reinforced earth wall systems (courtesy of Holtz, Christopher, and Berg 1995)

greatest economy is achieved using geotextile reinforcement. Facing elements are used to retain backfill material at the face of the wall, prevent erosion of steep faces, as well as for aesthetic reasons. Granular soils are normally used for soil backfill to meet stress transfer, durability, and drainage requirements. Soil backfill requirements for MSE walls are (Elias and Christopher 1997):

Sieve Size	Percent Passing
4 in. (102 mm)[1]	100
No. 40 (0.425 mm)	0 - 60
No. 200 (0.075 mm)	0 - 15
[1] For geosynthetics and epoxy coated material, the maximum particle size is (19 mm) 1 in.	

The soil backfill should be free of organic matter, and the plasticity index should not exceed 6.

Research is ongoing concerning use of poorly draining soils, such as clays and silts, as backfill for MSE (Chou and Wu 1993; Sabatini et al. 1997). Clayey backfills have lower drained shear strength than granular soils that results in larger lateral earth pressure against the wall, lower frictional resistance along the reinforcement for MSE walls that employs frictional reinforcement (such as geotextiles), and lower bearing value for MSE walls that employs passive reinforcement (such as geogrids). Clayey backfills have poor drainage and greater potential for corrosion of metallic reinforcements for MSE walls. Also, clayey backfills have the potential to undergo creep deformation that can lead to higher earth pressures and greater wall deformations. Despite these problems, clayey soil may be used as backfill material if suitable design procedures are followed (American Association of State Highway Transportation Officials (AASHTO) 1996; Sabatini et al. 1997). Using permeable geotextiles will dissipate excess pore water pressures and increase both the soil shear resistance along the potential failure plane and pullout resistance along the soil-geotextile interface. Nonwoven geotextiles offer high in-plane transmissivity and have excellent interface contact properties. However, nonwoven geotextiles have relatively low tensile strength, an important parameter in the design of geotextile reinforced structures. Therefore, a woven/nonwoven composite geotextile or high-strength nonwoven geotextile would be appropriate for reinforcement of poorly draining fills. The design of poorly draining fills with geotextiles used as reinforcement requires determination of the mechanical and hydraulic characteristics of the geotextile using in situ conditions including changes with time. Confinement may increase the stiffness and strength of the geotextile but decrease the in-plane transmissivity. This information is needed to determine the spacing of geotextiles for dissipation of excess pore water pressures and stability against reinforcement breakage and pullout (Zornberg and Mitchell 1992).

The advantages of MSE systems are: economical when compared to conventional structures; easy and rapid to construct and, regardless of wall height, the structure remains stable during construction; flexible and can tolerate

large lateral deformations and differential vertical settlements, thus allowing for the use of a lower factor of safety for bearing capacity than conventional more rigid structures; potentially better suited for earthquake loading because of flexibility and energy absorption capacity; polymeric reinforcements are stable under chemical and biological conditions that normally occur in soils; and a greater choice is available to meet aesthetic requirements than for conventional retaining walls, since facing elements play only a secondary structural role. The disadvantages of MSE systems are in the event of corrosion of metallic reinforcement, allowance must be made for polymeric reinforcement for decrease in strength due to abrasion during construction and creep (decrease in strength with time at constant load and soil temperature), and excavation behind MSE walls is restricted. Recent studies indicate that geosynthetic creep does not occur with granular backfill (Ketchart and Wu 1996). The design of MSE walls is covered in EM 1110-2-2502 (HQUSACE 1989); TM 5-818-8 (HQUSACE 1995b), Berg (1993), Mohoney et al. (1994), Elias and Christopher (1997), Elias (1997), and Sabatini et al. (1997).

Inflatable structure

When a levee must be raised to prevent overtopping, it may be feasible to use an inflatable structure (ASCE 1994b, Ennis 1997). Inflatable structures can be custom built several hundred feet long and to heights of up to 10 ft. The structures are constructed of flexible heavy duty vinyl, polyethylene, or geotextile sheet materials and can be filled with air or water. A variety of anchoring systems are employed to prevent the structures from rolling over under load. Inflatable structures are vulnerable to vandalism and damage from sharp objects carried by the river water. Some structures use ceramic chips as a coating to increase wearing resistance and protect from damage by sharp objects (Tam 1997). Another disadvantage of these structures is that they require a wide footprint (about three times the height) and would not be practical for many levees. Other structures such as the PORTADAM utilize a steel frame and waterproof membrane to retain water up to 9 ft deep with a smaller footprint (Portadam, Inc. 1996). A research study sponsored by the USACE is underway at the Department of Civil Engineering, Virginia Tech, to the study use of inflatable structures to raise levees (Duncan et al. 1997).

Lightweight material

While cellular confinement systems and inflatable structures can be used to raise the levee to prevent overtopping, this type of construction places a concentrated load on the levee and a distributed load on the underlying foundation. There may be cases where this causes problems involving slope stability, bearing capacity and/or settlement, and the use of lightweight artificial or waste materials would be beneficial. Lightweight materials including wood chips, tire chips (or shredded tires), and expanded polystyrene (or geofoam) blocks have been used for constructing embankments over soft foundations repair of slides,

subgrades for foundations, and extensions of airport runways (Flaate 1989, Transportation Research Board 1993, World Road Association 1997).

Lightweight materials are potentially useful for raising levees to prevent overtopping. Availability and cost would determine if they were competitive. Manufactured lightweight materials such as expanded polystyrene blocks are relatively expensive. Waste material such as wood chips and tire chips, which may be obtained almost free at the source, have a cost due to transportation and placement on site. Each case must be evaluated on its merits. The applicable type of construction for wood chips, tire chips, or expanded polystyrene blocks would be to raise the levee by enlarging the levee section (Humphrey et al. 1992, Drescher and Newcomb 1994, Edil and Bosscher 1992, Horvath 1995).

Wood chips have been used to construct embankments in Washington state. Wood chips have a compacted in-place unit weight of about 35 lb/cu ft. Recommendations include using only fresh wood fiber to prolong life of the fill (decomposition will occur with time), minimizing the volume of water entering the fill, and using 2 ft of soil cover to reduce decay and possibility of fire (Kilian and Ferry 1993). Information on the design and construction of lightweight fills using wood chips is available (World Road Association 1997).

Tire chips or shredded tires weigh about 45 lb/cu ft and are highly compressible under initial loading. A rebate may be given for use of a waste product which lowers cost. As mentioned previously, fires occurred in three thick (over 25 ft high) scrap tire embankments in 1995 which curtailed their use in highway applications for about 1 year until new guidelines were established. The guidelines, which limit the height of scrap tire embankments to 10 ft, are intended to minimize all possible factors which might contribute to internal heating. Lessons learned include processing tire chips by shearing rather than hammermill (less exposed steel wire and smaller surface area), limiting amount of exposed steel to less than 1 percent free steel by weight, avoiding covering tire fill with top soil or fertilizer (nitrogen or phosphate in soil/fertilizer may contribute to oxidation process leading to fires), and minimizing infiltration of water and air into the fills. Following these guidelines should lead to a conservatively designed tire chip fill that should not experience internal heating (Federal Highway Administration 1997).

Another technique which holds promise is constructing a wall with the same general configuration as shown in Figure 9 using mechanically stabilized lightweight (tire chip) backfill and geosynthetics (geogrids or geotextiles) as reinforcement. This would offer the advantage of a lightweight structure and free-draining backfill. This technique, proposed by Barrett (Fettig 1991), was constructed in 1995 by the Colorado Department of Transportation. Tire chips were used as backfill with geogrids for reinforcement behind a 70-ft-high retaining wall located alongside I-70 in Glenwood Canyon, Colorado (Humphrey 1996). Unfortunately, this was one of three tire chip fills with a thickness greater than 25 ft that experienced a catastrophic internal heating reaction. Although, as previously discussed, design guidelines are now available to minimize the possibility of heating in tire chip fills less than 10 ft high. No

subsequent structures utilizing tire chips as backfill and geosynthetics (geogrids or geotextiles) as reinforcement have been built. Limited data are available on pullout resistance of tire chips and geogrids (Bernal, Lovell, and Salgado 1996).

Expanded polystyrene blocks, weighing about 1.25 lb/cu ft, offer the greatest advantage in weight saving. They are well-suited to raising levees which are founded on compressible soils. The blocks should be placed on a level surface and must be protected against flotation and petroleum spills. This can be accomplished by leveling the foundation surface and placing a membrane and soil cover over the blocks. Anchors can be provided as needed. Expanded polystyrene is a standard building material that has been used in the construction industry for over 50 years in various applications such as perimeter insulation, block inserts, drywall backer, concrete void fillers, etc. Expanded polystyrene is inert, nonnutritive, and highly stable and therefore will not decompose, decay, or produce undesirable gases or leachates. Samples retrieved from existing road fills in Norway show no signs of decay or strength reduction after 20 years. Blocks are routinely treated (at little additional cost) to be flame retardant and to deter insect infestation. A high groundwater table in the levee will expose the expanded polystyrene blocks to water on a long-term basis. Field data from expanded polystyrene blocks below the groundwater table which have been excavated from construction sites in Norway show that over a 20-year period the increase in water is about 9 percent by volume or about a 6-lb/cu ft gain in weight for the blocks. This relatively small, gradual increase in unit weight of the expanded polystyrene blocks will cause a small increase in the amount of consolidation settlement of a soft foundation (Flaate 1989).

Expanded polystyrene blocks have been used to construct hundreds of embankments in Norway, Japan, and the United States (Horvath 1995). In 1993, a value engineering proposal was made to the USAED, Galveston, to use expanded polystyrene blocks to raise the hurricane flood protection levee at Port Arthur, Texas (USAED, Kansas City 1993). This proposal, which was a lower cost than the original design (driving steel sheetpiling) and would have been the first use of expanded polystyrene blocks to raise a levee, was not adopted. Expanded polystyrene blocks were used to reconstruct a flood levee along the Thorne River in Humberside, England (Sanders 1996, Horvath 1996). The design and construction of expanded polystyrene block embankments is given by Horvath and others (Horvath 1994, 1995; Sanders and Snowdon 1993; Negussey 1997). There is a World Wide Web site dedicated to geotechnical applications of expanded polystyrene blocks (http:/www.geocities.com/~geofoam/).

Overview of innovative overtopping prevention

A synopsis of innovative methods to raise the levee to prevent overtopping is given in Table 3. Cellular confinement systems are a promising method which offer proven construction techniques, wave protection (if needed), and no disadvantages. Mechanically stabilized earth may prove useful as ongoing research allows the designer to use clayey (on site) backfill and geotextile

Table 3
Overview of Innovative Rehabilitative Methods for Overtopping Prevention (Methods to Raise the Levee)

Rehabilitative Method	Applicable Conditions	Advantages	Disadvantages
Cellular confinement systems	Applicable for most levees	Good construction techniques (notched grid, sand/bentonite mix, concrete fill where required	None
Mechanically stabilized earth	Applicable for most levees	Economical compared to conventional structures	Metal reinforcement may corrode
		Flexible structures	Creep may occur with clayey backfill
Inflatable structure	Temporary structure	Structures can be rented or purchased	Low wave protection
		Easily transported and installed	Subject to vandalism and damage from sharp objects carried by river
Lightweight Material			
Wood chips	Only available in a few geographic regions	Compacted in-place unit weight 35 lb/cu ft	Settlement may equal 10 percent of compacted thickness
Tire chips or shredded tires	Widely available as waste product	Compacted in-place unit weight 45 lb/cu ft	Highly compressible, if additional load (such as a road) is placed on the section
	Fires in fills in 1996 limited current use to fills less than 3 m (10 ft) high		
Mechanically stabilized (tire chip) backfill	Widely available as waste product	Compacted in-place unit weight 45 lb/cu ft	Restricted to structures less than 3 m (10 ft) high
		Low compressibility	
Expanded polystyrene blocks	Well-suited for use over compressible foundations	Superlight with in-place unit weight 1.25 lb/cu ft	Relatively high costs
		Only 6-lb/cu ft increase in unit weight under high groundwater table conditions	Must protect against flotation and petroleum spills

reinforcement. Inflatable structures may offer advantages as temporary structures in low wave environments in certain instances.

When conditions dictate that a minimum load should be placed on the levee and/or foundation when raising the levee, lightweight materials may be used. When the levee is located in a geographic region where wood chips are available as a waste material, wood chips with a unit weight about one-fourth the unit weight of soil could be used to construct an enlarged levee section and raise the levee. Tire chips, widely available as a waste product, with a unit weight about one-third the unit weight of soil could be used to construct an enlarged levee section which would be more compressible under load. For cases where the

increase in height of the levee was 10 ft or less, mechanically stabilized tire chip backfill could be used to construct a more compact double-wall structure (as depicted in Figures 8 and 9) which could be less compressible. In situations where the levee and/or foundation properties were such that it was desired to place as little additional load as possible on the levee, expanded polystyrene blocks with a unit weight about one-twentieth the unit weight of soil could be used to enlarge the levee section.

3 Current and Wave Attack

Background

During times of flood, when the river is beyond its banks and the water is up against the levee, the riverside slope of the levee is subject to current action and possibly wave attack. The current exerts a tractive shear stress (force per unit area) as the water flows past the levee. This stress may cause erosion of the levee unless it is properly protected. The tractive shear stress on the riverside slope of the levee is directly proportional to the depth of the water and is a minimum at the water's edge and a maximum near the riverside toe of the levee. Therefore, although the levee is not in danger of overtopping due to erosion from current flow, some erosion may occur at the riverside toe of the levee which could be repaired after the flood receded.

During flood, waves from wind and/or vessels may impinge upon the levee. The wind exerts drag on the water surface which generates waves. The magnitude and frequency of wind-generated waves are dependent on wind velocity, duration of the wind, fetch distance, orientation and surface area of the exposed water surface, and depth of the water. During flood events when the water is up against the levee, the fetch distance is increased and the potential exists for wind-generated waves (depending on wind velocity, duration of the wind, etc.) to cause erosion of the levee slope. Trees between the river and levee may provide some wave protection (model tests indicate wave attenuation through trees with branches without foliage is about 15 percent (Markle 1979)).

Waves are generated from vessels in the form of ship waves and propeller wash. Ship-generated waves and propeller wash may cause erosion of streambanks in channel bends, lock entrances, mooring and fleeting areas. Significant erosion of levees does not occur because in times of flood, operation of vessels is restricted (USAED, Huntington 1980; Bhowmik and Schicht 1980).

The line of application of waves acting on the riverside slope of the levee will vary with the river stage. The potential for damage will be greatest when the river stage is at a maximum, i.e., close to the top of the levee, where wave action and runup can cause erosion, and possible overtopping, and breaching of the levee.

As shown previously in Table 1, conventional methods to protect the levee against current and wave attack include vegetation, revetment (riprap, concrete rubble, soil cement blocks, used auto tires, etc.) and gabions. Innovative methods of current and wave protection include reinforced grass (current only), concrete block system, and soil cement/RCC.

Reinforced Grass

As discussed in Chapter 2, reinforced grass would be a viable option for protection of the riverside levee slope against current action when the duration of flow during flood was less than 2 days, poor cover of newly sown grass during the first growing season was acceptable and root survival during drought was not a problem. These restrictions limit the usefulness of reinforced grass as protection of the riverside levee slope against current action.

Reinforced grass is not substantial enough to offer protection against wave action (Hewlett, Boorman, and Bramley 1987).

Concrete Block System

Concrete block systems, discussed in Chapter 2, would offer adequate protection against current flow up to 25 ft/sec or a hydraulic shear stress of 15 lb/sq ft (Clopper and Chen 1988).

During the USACE Low-Cost Shore Protection Program, various concrete block systems (Gobi blocks, Turfblocks, hollow concrete building blocks, Sandgrabber blocks, and Nami rings) were tested at several sites representing a variety of environments (USACE 1981a,b). Results of the study indicated that hand-placed (articulated) concrete block revetments will survive in areas with waves less than 2 ft high and concrete blocks glued to a filter fabric to form large mats should survive 3-ft-high waves (Combe et al. 1989). More recent concrete block revetment systems threaded with polyester or galvanized steel cables to form prefabricated mattresses should prove stable under higher wave environments. Although experiments have been done on specific concrete block systems (Delft Hydraulics Laboratory and Delft Soil Mechanics Laboratory 1983), available design guidance does not cover allowable wave heights for different types of block revetments placed on various slopes (Bezuijen, Breteler, and Burger 1990; HQUSACE 1995a). Concrete block systems have relatively smooth faces which could lead to significantly higher wave runup and possible overtopping.

In summary, concrete block systems should provide adequate protection against current flow (in excess of 25 ft/sec) and low wave (up to 2 ft) environments along levees. For higher wave environments, concrete block revetment systems threaded with polyester or galvanized steel cables to form prefabricated mattresses should prove stable (Naghavi and Allain 1990).

Soil Cement/RCC

As discussed in Chapter 2, soil cement or RCC may be placed and compacted in stairstep horizontal layers or by plating a single layer placed parallel to the slope. Erosion of soil cement and/or RCC from current attack can occur due to the hydraulic shear stress exerted by the flowing water; abrasive action from sand, gravel, or other waterborne debris; and cavitation from surface imperfections at flow velocities as low as 40 ft/sec (HQUSACE 1993b, ASCE 1994a). If coarse sediment was present in the current flow, an RCC mixture with a low water-cement ratio and larger-size aggregates would provide erosion resistance equal to conventional concrete with similar ingredients (McLean and Hansen 1993). The relatively rough surface of soil cement and RCC would result in cavitation erosion if flow velocities exceed 40 ft/sec. As discussed in Chapter 2, soil cement would offer adequate protection during the lifetime of the levee for nonsediment laden flow velocities up to 20 ft/sec (equivalent to a hydraulic shear stress of 45 lb/sq ft).

RCC could be used for flows with or without coarse sediment present and for flow velocities up to 35 ft/sec (upper limit of available experimental data). If coarse sediment was present in the current flow which would abrade the RCC protection, RCC with increased strength and larger-size aggregates could be used (McLean and Hansen 1993). If flow velocities exceeded 40 ft/sec and the cumulative amount of predicted cavitation erosion was unacceptable, a conventional concrete topping or facing (with a formed or screeded smooth surface) could be placed over the RCC to prevent cavitation erosion (HQUSACE 1993b, ASCE 1994a).

Soil cement or RCC placed and compacted in stairstep horizontal layers or by plating a single layer placed parallel to the slope has been used as wave protection for dams for many years (USBR 1987). While the plating method uses less material, it is difficult to install on steep (20 percent or greater) slopes and the smooth face results in greater wave runup (Bingham, Schweiger, and Holderbaum 1992; PCA 1992). The stairstep method is easier to construct and provides better wave energy dissipation due to the stepped configuration. Soil cement should provide adequate wave protection when the stairstep method of construction is used. RCC should be considered whenever the single-layer construction technique is to be employed. For a slight increase in cost, RCC will provide increased strength and greater resistance against erosion (McLean and Hansen 1993).

Overview of Innovative Current and Wave Attack Protection Methods

A synopsis of innovative rehabilitative methods for current and wave protection of levee riverside slope is given in Table 4. Reinforced grass has limited usefulness for current protection because it would be restricted to

conditions where the duration of flow during flood was less than 2 days, poor cover of newly sown grass during the first growing season was acceptable, and root survival during drought was not a problem. Concrete block systems would offer adequate protection against current flow up to 25 ft/sec or a hydraulic shear stress of 15 lb/sq ft. Soil cement would offer adequate protection during the lifetime of the levee for nonsediment laden flow velocities up to 20 ft/sec (equivalent to a hydraulic shear stress of 45 lb/sq ft). RCC could be used for flows with or without coarse sediment present, and flow velocities up to 35 ft/sec. If flow velocities exceeded 40 ft/sec, a concrete topping or facing placed over the RCC would prevent cavitation erosion.

Reinforced grass is not substantial enough to offer protection against wave attack. Concrete block systems should provide adequate protection against low wave (up to 2 ft) environments along levees. For higher wave environments, concrete block revetment systems threaded with polyester or galvanized steel cables to form prefabricated mattresses should prove stable. Soil cement or RCC placed and compacted in stairstep horizontal layers or by plating a single layer placed parallel to the slope will provide adequate wave protection for levees. While the plating method uses less material, it is difficult to install on steep (20 percent or greater) slopes and the smooth face results in greater wave runup. Soil cement should provide adequate wave protection when the stairstep method of construction is used. RCC should be considered whenever the single-layer construction technique is to be employed.

Table 4
Overview of Innovative Rehabilitative Methods for Current and Wave Protection of Levee Riverside Slope

Rehabilitative Method	Applicable Conditions	Advantages	Disadvantages
Current Protection			
Reinforced grass	Limited usefulness since flow duration must be less than 2 days[1]	Low capital costs	Susceptible to damage prior to development of root system
	Flow velocity depends on duration (Figure 3)	Reduces risk of localized failure	More expensive to maintain
			Root system may not survive drought
Concrete block system	Flow velocities up to 25 ft/sec (hydraulic shear stress of 15 lb/sq ft)	Hydraulic stability is independent of flow duration	If blocks are not cabled together, system could fail if one block is lost
Soil cement	Flow velocities up to 20 ft/sec or hydraulic shear stress of 45 lb/sq ft (upper limit of available experimental data)	Relatively low cost	Not applicable if coarse sediment present in flow
	Nonsediment laden flow	Speed of construction	Must provide drainage to relieve excess hydrostatic pressure
		Requires no coarse aggregate for mix	
RCC	Flow velocities up to 35 ft/sec (upper limit of available test data)	Relatively low cost	Relatively rough surface susceptible to cavitation erosion if flow velocity exceeds 40 ft/sec
	Nonsediment laden flow[2]	Speed of construction	Must provide drainage to relieve excess hydrostatic pressure
	Flow velocities over 40 ft/sec[3]	High strength and erosion resistance	
Wave Protection			
Reinforced grass	Not substantial enough for wave protection	NA	NA
Concrete block systems:			
Hand-placed blocks	Will survive 2-ft-high waves	Flexible and tolerates irregular settlement	System could fail if one block is lost
		Conforms to irregular geometry	Smooth face gives higher wave runup

(Continued)

[1] This time period could possibly be extended by using more flood resistant grass and/or providing additional anchorage (helical and/or duckbill depending on the soil conditions) to the reinforced grass system.
[2] If coarse sediment was present in the flow, RCC with increased strength and larger-size aggregates could be used (McLean and Hansen 1993).
[3] If flow velocities exceeded 40 ft/sec and the cumulative amount of cavitation erosion was unacceptable, a conventional concrete topping or facing (with a formed or screeded surface) could be constructed over the RCC to extend the range of acceptable flow velocity (HQUSACE 1993b, ASCE 1994a).

Table 4 (Concluded)			
Rehabilitative Method	**Applicable Conditions**	**Advantages**	**Disadvantages**
		Wave Protection	
Concrete block systems:			
Blocks glued to filter fabric	Will survive 3-ft-high waves	Bridge over voids	Smooth face gives higher wave runup
		Easy to place in mats under water using spreader bars	
Blocks cabled together with polyester or steel	Will survive higher waves	Bridge over voids	Smooth face gives higher wave runup
		Easy to place in mats under water using spreader bars	
Soil cement	Stairstep method of construction should be used	Relatively low cost	Must provide drainage to relieve excess hydrostatic pressure
	Should provide adequate wave protection	Speed of construction	
		Requires no coarse aggregate for mix	
RCC	Consider for use with single-layer construction	Relatively low cost	Difficult to install on steep slopes
	Should provide adequate wave protection	Speed of construction	Smooth single layer gives higher wave runup
		High-strength and erosion resistance	Must provide drainage to relieve hydrostatic pressure

4 Surface Erosion Due to Rainfall

Background

Rainfall erosion begins when raindrops strike the surface of the levee and detach soil particles by splash. The erosive potential of rainfall depends on the raindrop fall velocities, size distribution, and total mass at impact. Runoff occurs when the rainfall intensity exceeds the infiltration rate of the soil. Once the soil is detached by the raindrop impact, sheet and rill erosion occur. Sheet erosion is the removal of a fairly uniform layer of soil by the action of raindrop impact and runoff shear stress. Soil removal is uniform only from raindrop splash. Once runoff starts, rill erosion soon begins and erosion is no longer uniform. Rills may develop into gullies (erosional features which cannot be removed by normal soil cultivation). There are four major factors which contribute to rainfall erosion (Perry 1975):

a. The nature of the rainfall as given by its intensity, duration, drop size distributions, drop velocity, and impact energy.

b. The properties of the soil affecting infiltration, erodibility, sediment transport, and deposition.

c. The steepness and length of the slope.

d. Cover provided by plants and residues and/or chemical erosion prevention techniques.

Rainfall erosion is possible on the riverside slope, crest, and landside slope of the levee. Normally, grasses and herbaceous vegetation provide a canopy which prevents the raindrops from striking the surface of the levee and detaching soil particles by splash and a root system which resists sheet and rill erosion (Coppin and Richards 1990, Gray and Sotir 1996). However, there may be cases where, due to drought or season of the year, the condition of the canopy and/or root system is poor and does not provide sufficient protection against rainfall erosion and additional measures may be required (Agassi 1996). For the special case of

dispersive soil, vegetation will not provide protection against rainfall erosion and some stabilization technique, such as lime modification or gravel cover, may be required (Perry 1979).

As shown previously in Table 1, conventional methods to protect the levee against surface erosion due to rainfall include vegetation and chemical stabilization. Turf reinforcement mats are an innovative method of rainfall erosion protection.

Turf Reinforcement Mat

Rolled erosion control products (RECP) are temporary degradable blankets or long-term nondegradable mats designed to reduce soil erosion. A temporary degradable RECP blanket, composed of a lightweight polymer net(s) and a bedding of polymer or organic materials, is used to retain moisture, seeds, and soil to promote vegetation growth. The polymer materials used in the blanket are typically not stabilized against ultraviolet light and degrade over time, with design lives between 6 months to 5 years. A long-term nondegradable RECP turf reinforcement mat, composed of ultraviolet stabilized, synthetic fibers, nettings and/or filaments processed into three-dimensional (3-D) reinforcement matrices, is used to reinforce the vegetation root mass where design velocities and shear stresses exceed the limits of vegetation. Turf reinforcement mats are designed to furnish erosion protection for the design life of the project (Allen 1997; Holtz, Christopher, and Berg 1995). Although developed for protection against hydraulic shear stress due to water flowing over soil, turf reinforcement mats would offer adequate protection against rainfall erosion by eliminating raindrop impact on the soil and providing protection against erosion due to runoff. Computer programs for the design of slope protection are available from companies which market turf reinforcement mats (American Excelsior Company 1996, Lancaster 1995, Synthetic Industries 1995, Sprague 1997). For turf reinforcement mats, the limiting flow velocity would vary with flow duration, similar to reinforced grass shown in Figure 3. The flow duration would equal the duration of the rainfall at the levee location.

5 Through-Seepage

Background

As mentioned previously in Chapter 1, recent floods have subjected levees to long periods of water retention with resulting through-seepage. When seepage emerges on the landside slope of the levee, it can cause sloughing of the slope and/or lead to piping (internal erosion) of levee materials. As shown previously in Table 1, conventional methods to protect the levee against through-seepage include toe drains and conventional chimney drains. Biopolymer chimney drains are an innovative method of through-seepage control.

Biopolymer Chimney Drain

The biopolymer drain was recently developed in response to the need for an economical method for constructing drains in existing embankments. This system uses basic slurry trench technology, but instead of bentonite slurry, a natural or synthetic organic compound is used to maintain an open trench. Natural biopolymers may come from plant or tree gums or algae. Synthetic biopolymers are generally cellulosic derivatives. Once the trench is excavated, the drain material is placed using a tremie operation or sliding the backfill down the slope of previously placed backfill to displace the slurry and minimize segregation. Wells can be inserted and pipe laterals placed under the slurry. Once the installation is complete, the trench can be sluiced with a dilute chlorine solution (50-50 water/Clorox®) to break the polymer strands and facilitate pumping the slurry from the trench to develop the drain (Tallard 1992a,b; Day and Ryan 1992; Perry 1993; Ata and O'Neill 1997).

Biopolymer drains have been utilized in the United States for about 10 years for environmental cleanups (Day and Ryan 1992). The USBR recently rehabilitated 15 miles of Central Arizona Project dikes near Phoenix using a synthetic biopolymer slurry trench and tremie placement of sand to form a chimney drain (Anonymous 1996, Bliss 1994, 1995). Subsequently, a chimney drain was constructed using a synthetic biopolymer slurry to excavate a trench at Hays Creek Dam in New Zealand (Jairaj and Wesley 1995). Both of these

applications illustrate the usefulness of biopolymer slurry to construct chimney drains for through seepage control in levees.

6 Underseepage

Background

As shown previously in Table 1, existing methods to protect the levee against underseepage include conventional toe trenches and cutoffs, riverside blankets, landside seepage berms, and pressure relief wells. Biopolymer toe trenches and jet grouted cutoffs are innovative methods of underseepage control.

BioPolymer Toe Trench

When a levee is located on pervious deposits overlain by little or no impervious material, a partially penetrating toe trench with a perforated collector pipe can improve seepage conditions at the levee toe, as shown in Figure 11 and discussed in EM 1110-2-1913 (HQUSACE 1978). Open trench excavations can be made above the groundwater table, but below the groundwater table, a dewatering system is required. Since dewatering is a costly procedure, the slurry trench method of construction is often used. The biopolymer slurry trench method offers unique advantages for constructing a toe trench with collector pipe.

As previously discussed in Chapter 5, the biopolymer drain was recently developed in response to the need for an economical method for constructing drains in existing embankments. Once the trench is excavated, perforated pipe can be positioned and the backfill material placed using a tremie operation or sliding the backfill down the slope of previously placed backfill to displace the slurry and minimize segregation. Once the installation is complete, the trench can be sluiced with a chlorinated water solution to break (reduce) the viscosity and facilitate pumping the slurry from the trench to develop the drain (Tallard 1992a,b; Day and Ryan 1992; Perry 1993).

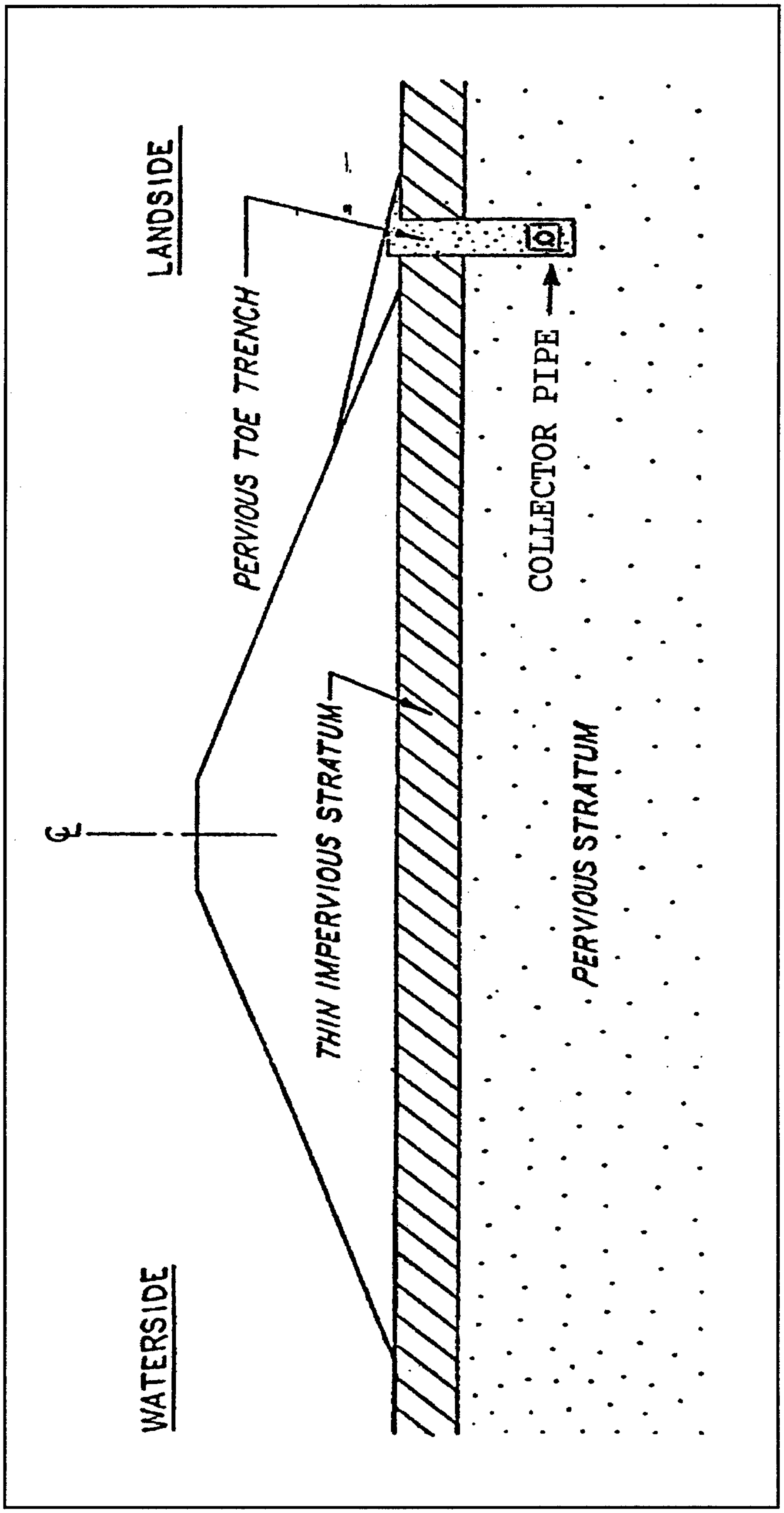


Figure 11. Partially penetrating pervious toe trench with collector pipe

Jet Grouted Cutoff

A cutoff beneath a levee through the pervious foundation is the most positive method of underseepage control as discussed in EM 1110-2-1901 and EM 1110-2-1913 (HQUSACE 1986, 1978). However, completely cutting off pervious strata 80 to 200 ft deep along an extensive reach of levee is not economically feasible. Partially penetrating cutoffs will not reduce seepage significantly unless the cutoff penetrates 95 percent or more of the pervious aquifer. However, shallow cutoffs extending through relatively thin layers of pervious material which are underlain by more impervious strata are an effective way of reducing underseepage (USAEWES 1956, HQUSACE 1997).

Normally, the cutoff is located under or near the riverside toe of the levee and consists of a compacted backfill trench or soil-bentonite slurry trench cutoff. If it is not feasible to construct a conventional soil-bentonite slurry trench cutoff located at the riverside toe of the levee, a jet grouted cutoff could be constructed through the pervious foundation underneath the center of the levee as shown in Figure 12. Jet grouting is a general term used to describe a construction method which utilizes a high-speed fluid to cut, replace, and then mix the native soil with a cementing material, often a water-cement grout. Jet grouting is uniquely suited for constructing cutoff walls to control underseepage, because it can begin at the interface between the levee and foundation and terminate at the bottom of the pervious foundation layer. Cutoff walls may be formed by a single line of columns, double line of columns, or panel wall as shown in Figure 13. A pilot hole is used to maintain vertically of the columns at depth. Design and construction of jet grouted cutoffs are given in several sources (Guatteri, Mosiici, and Altan 1988; Bruce 1988; Kauschinger, Perry, and Hankour 1992; Kauschinger, Hankour, and Perry 1992; Bell 1993; Welsh and Burke 1995).

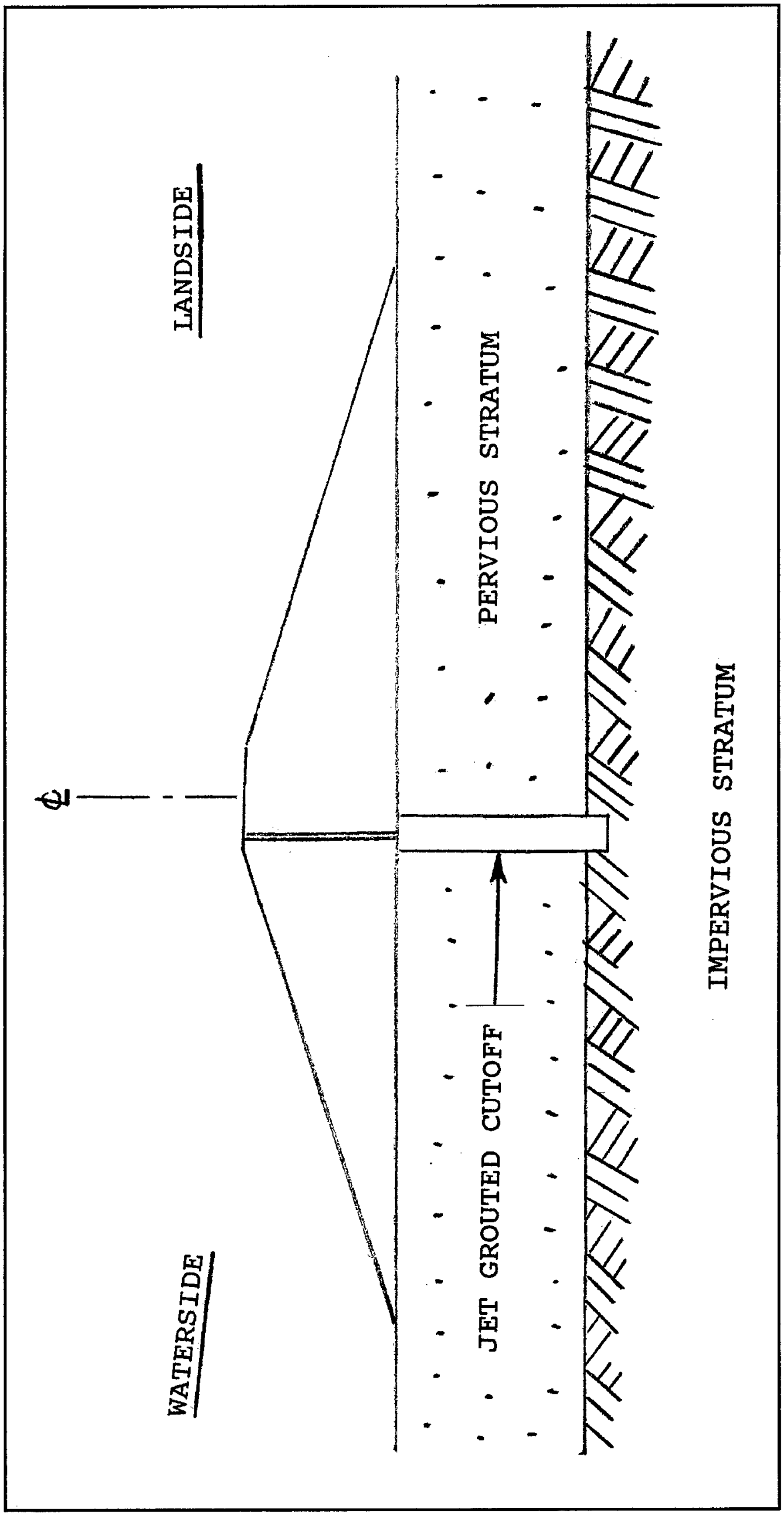


Figure 12. Jet grouted cutoff wall under levee

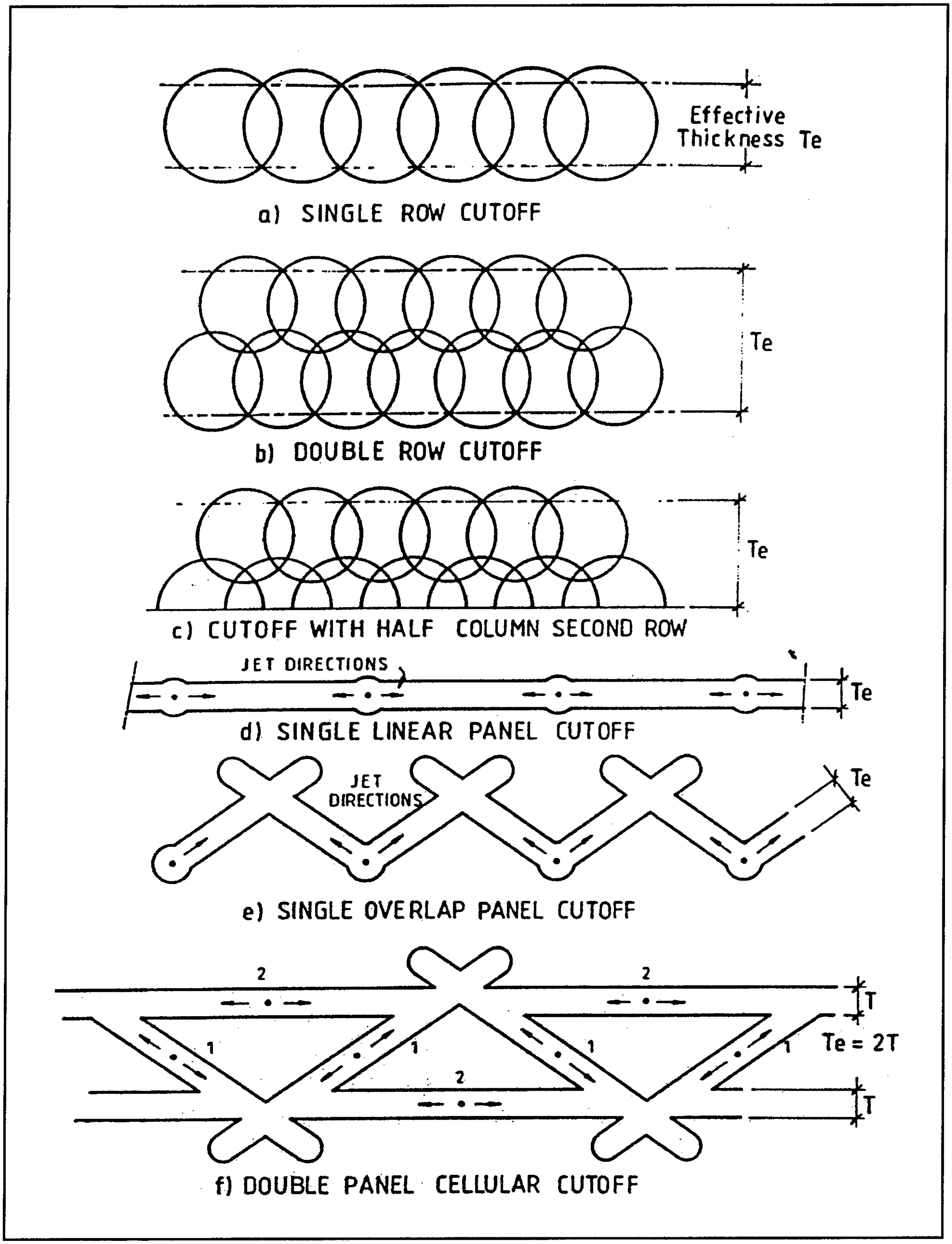


Figure 13. Typical columns and panel layouts for jet grouted cutoffs (courtesy of Bell 1993)

7 Slope Instability

Background

Shallow slides, as shown in Figure 14, frequently occur along levees. While most of these slides are not considered an immediate threat to the safety of the structure, the slides must be repaired to avoid breaching of the levee during flood. In one year (1979), the USAED, Vicksburg, repaired 41 slides. Slides typically occur in montmorillonitic clay with a plasticity index greater than 40 and liquid limit greater than 60. Weathering (repeated cycles of desiccation and wetting) produces shrinkage cracks, and rainfall runoff may fill the cracks more quickly than the cracks can swell closed. If this occurs, the hydrostatic force due to the water in the cracks exerts a lateral thrust, which together with the decreased shear strength caused by softening of exposed surfaces in the cracks, may lead to shallow slope failure (Bromhead 1992). Slope failures typically occur when an intense rainfall closely follows a period of several months of low rainfall (McCook 1993, 1997; Sills and Templeton 1983; Templeton, Sills, and Cooley 1984). Slides occur 2 to 35 years (with an average of 18 years) following construction of the levee. The depth of the slides, from 4 to 8 ft normal to the slope, coincides with the depth of desiccation (5 to 7 ft). Other agencies such as the Natural Resources Conservation Service, Texas State Highway Department, and Louisiana Department of Transportation have experienced problems with shallow slides in embankments constructed of plastic clay (U.S. Department of Agriculture Soil Conservation Service 1988; Cuenca and Wright 1988; Wright and Cuenca 1986; Juran et al. 1989; and Burns et al. 1990).

As shown previously in Table 1, existing methods to protect the levee against slope instability include drainage, removal and replacement of soil (slope flattening and benching), conventional restraint structures, and chemical treatment by mixing in place (cement, lime, flyash, etc.). The method adopted for use depends on several factors such as right-of-way, available borrow material, maximum steepness of slope for maintenance, costs, etc. Innovative methods to correct slope instability include reinforced soil slope, soil nailing, pin piles, stone-fill trenches, randomly distributed synthetic fibers, restraint structure, geosynthetic drainage system, lime-fly ash injection and anchored geosynthetic system.

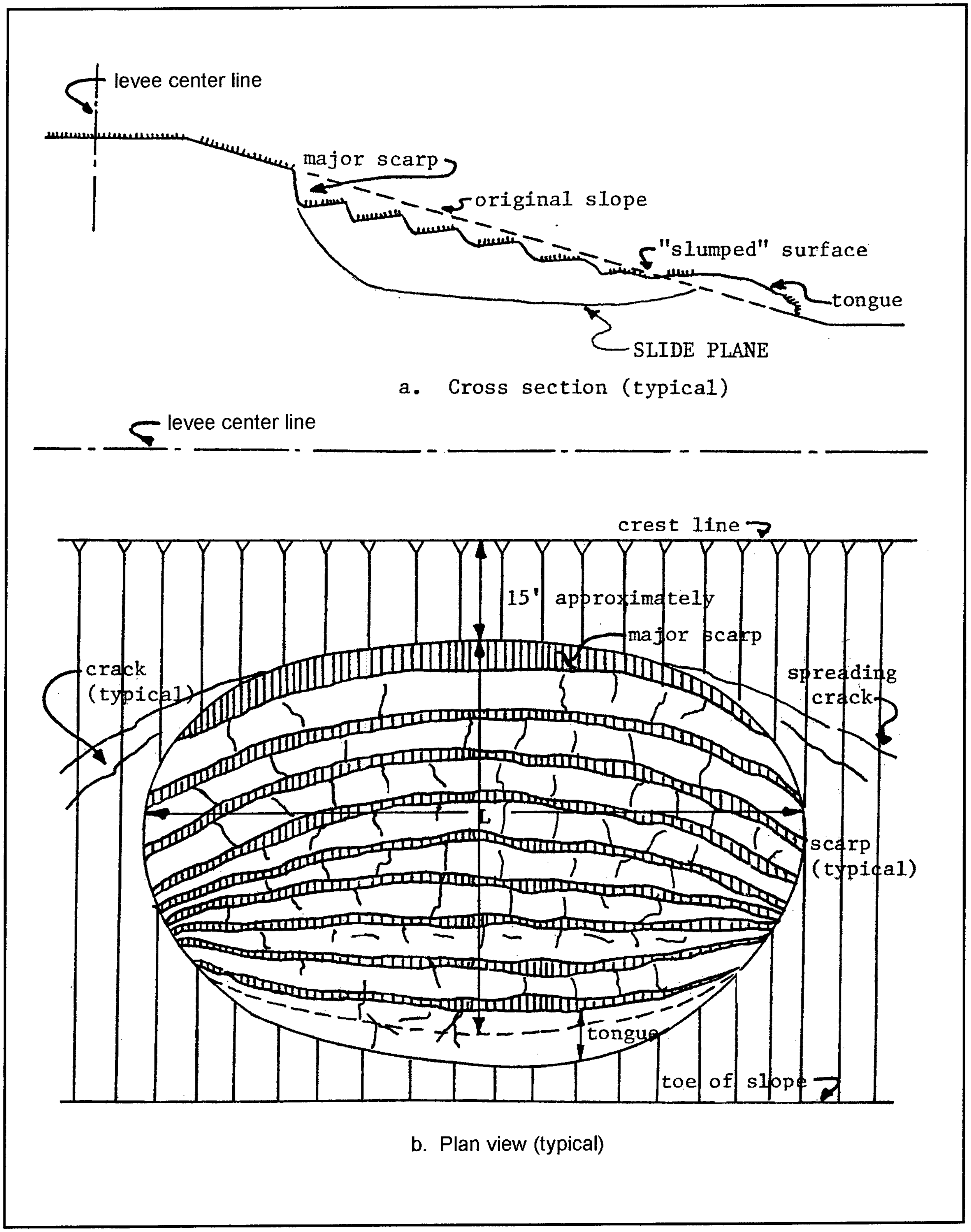


Figure 14. Typical slide in Mississippi River levee (after Sills and Templeton 1983)

When the mechanism of slope instability involves shrinkage cracks, a workable slope rehabilitation method must do one of the following (inclusion of a method in a particular category does not guarantee it will work but rather it has been used for that application):

a. Prevent the formation of shrinkage cracks (randomly distributed synthetic fibers).

b. Collect the rainfall runoff which enters the cracks and carry it away from the slope, thus preventing buildup of hydrostatic pressure and loss of shear strength (geosynthetic drainage system).

c. Maintain stability of the slope (or in some cases, restore the stability of a failed slope) with shrinkage cracks (pin piles, stone-fill trenches, restraint structure, lime-fly ash injection).

d. Prevent the formation of shrinkage cracks and maintain stability of the slope (reinforced soil slope).

When the mechanism of slope instability does not involve shrinkage cracks, soil nailing or an anchored geosynthetic system may work.

Reinforced Soil Slope (RSS)

As mentioned previously in Chapter 2, reinforced soil has been used for the past 20 years to repair slopes (Bergado et al. 1994; Murray 1985; Jewell 1985; Jewell, Paine, and Woods 1985; Bonaparte, Holtz, and Giroud 1987). The failed slope is excavated to a depth behind the failure plane and rebuilt using reinforcement as shown in Figure 15 (Jewell 1985; Hopkins et al. 1988). A drainage system is included if necessary (Hausmann 1992). Reinforcement used includes geotextiles, geogrids, and fibers (Oliver 1985, Holtz and Schuster 1996). Whereas galvanized steel reinforcement is commonly used for mechanically stabilized walls (Chapter 2), polymeric (geogrids and geotextiles) reinforcement is used for most RSS applications (Elias 1997). Field tests were conducted from 1984 to 1989 by the Federal Highway Administration of four RSS to verify design methods (Christopher, Bonczkiewicz, and Holtz 1994). A recent project in California used 29 RSS with one section 65 ft high and 1,800 ft long (Miyake et al. 1993). The USBR used geogrids to steepen the downstream slope of Davis Creek Dam in central Nebraska (Engemoen and Hensley 1989, Engemoen 1993). The USAED, Memphis, has been using geogrids to repair slides on levee slopes for several years as shown in Figures 16 and 17 (Abernathy 1994). Properties of geotextiles and geogrids typically used to construct RSS are given in the "Specifier's Guide" published annually by the Industrial Fabrics Association International (1996).

Advantages and disadvantages of RSS are given in Table 5 (Sabatini et al. 1997; Elias and Christopher 1997). The primary advantage of RSS is that it can utilize existing levee soils and rebuild to steeper slopes with a resulting cost

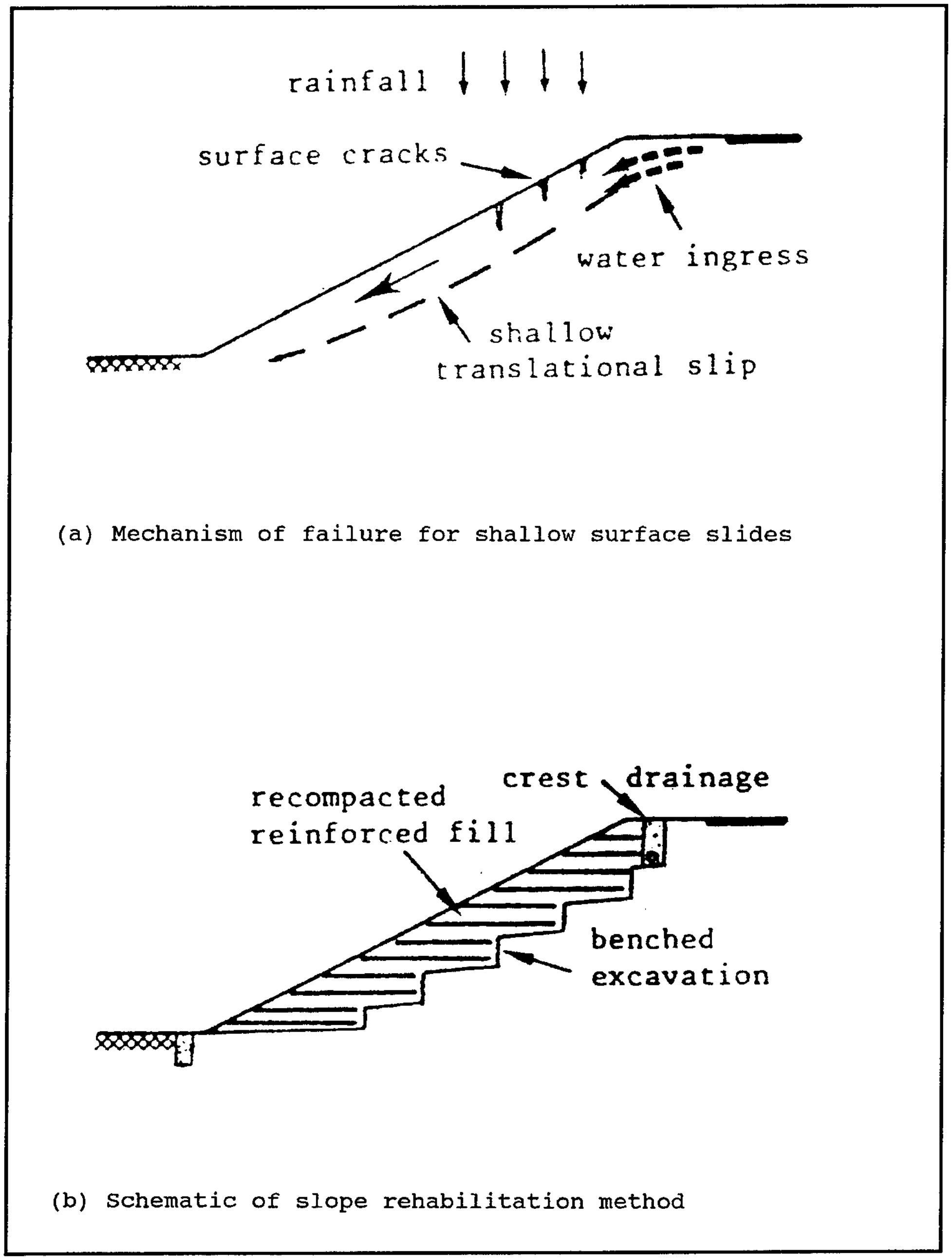
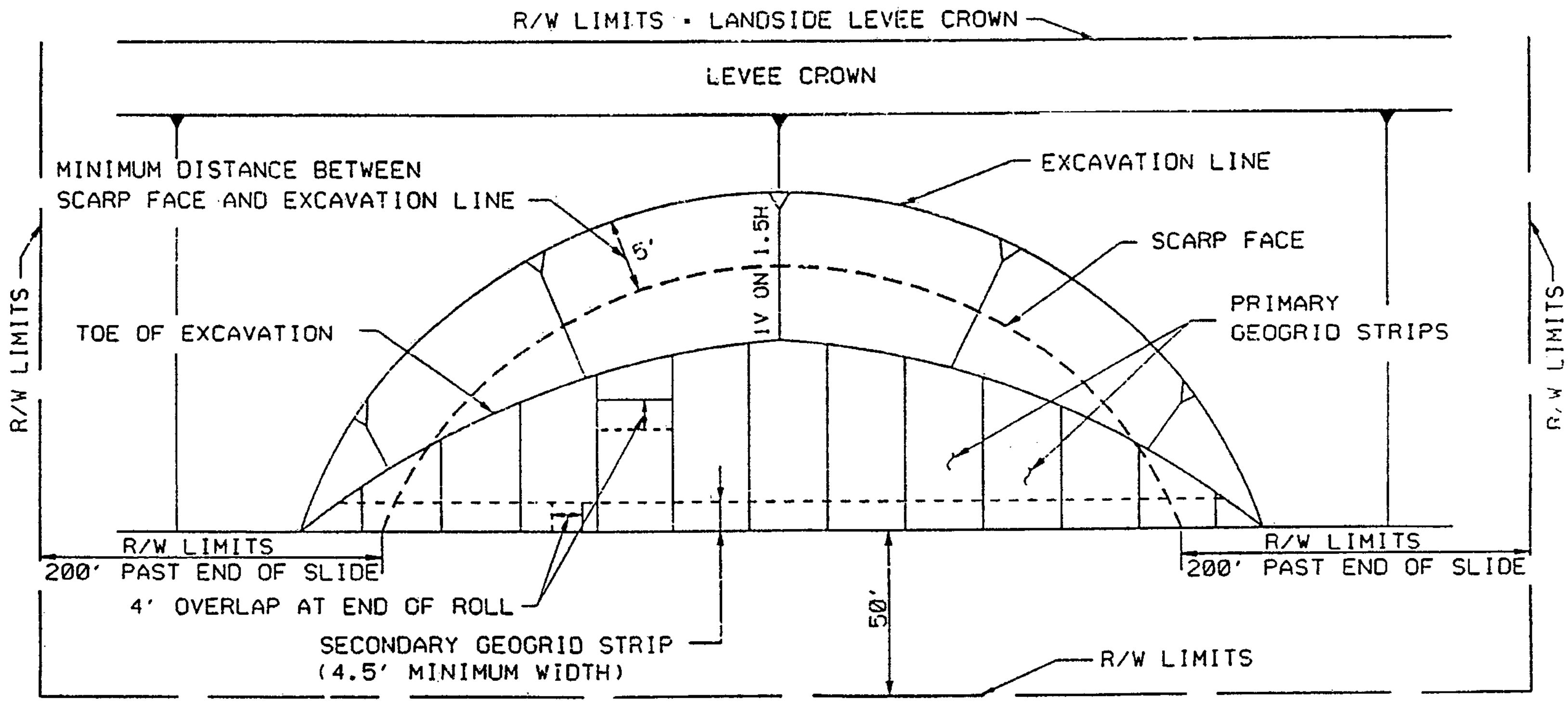


Figure 15. Use of geosynthetics to repair slopes (courtesty of Jewell 1985)

savings and utilization of less right-of-way. The primary disadvantage of RSS is that some uncertainty exists as to the effect of construction installation damage and durability (chemical and biological degradation with time) on the tensile strength of polymeric reinforcements (Elias and Christopher 1997; Elias 1997; Salman et al. 1997; Allen and Elias 1996; Fuller 1993; Hausmann 1990; Jewell and Greenwood 1988; Richardson and Wyant 1987).

SLIDE REPAIR - SIDE SLOPE

PLAN VIEW
NTS

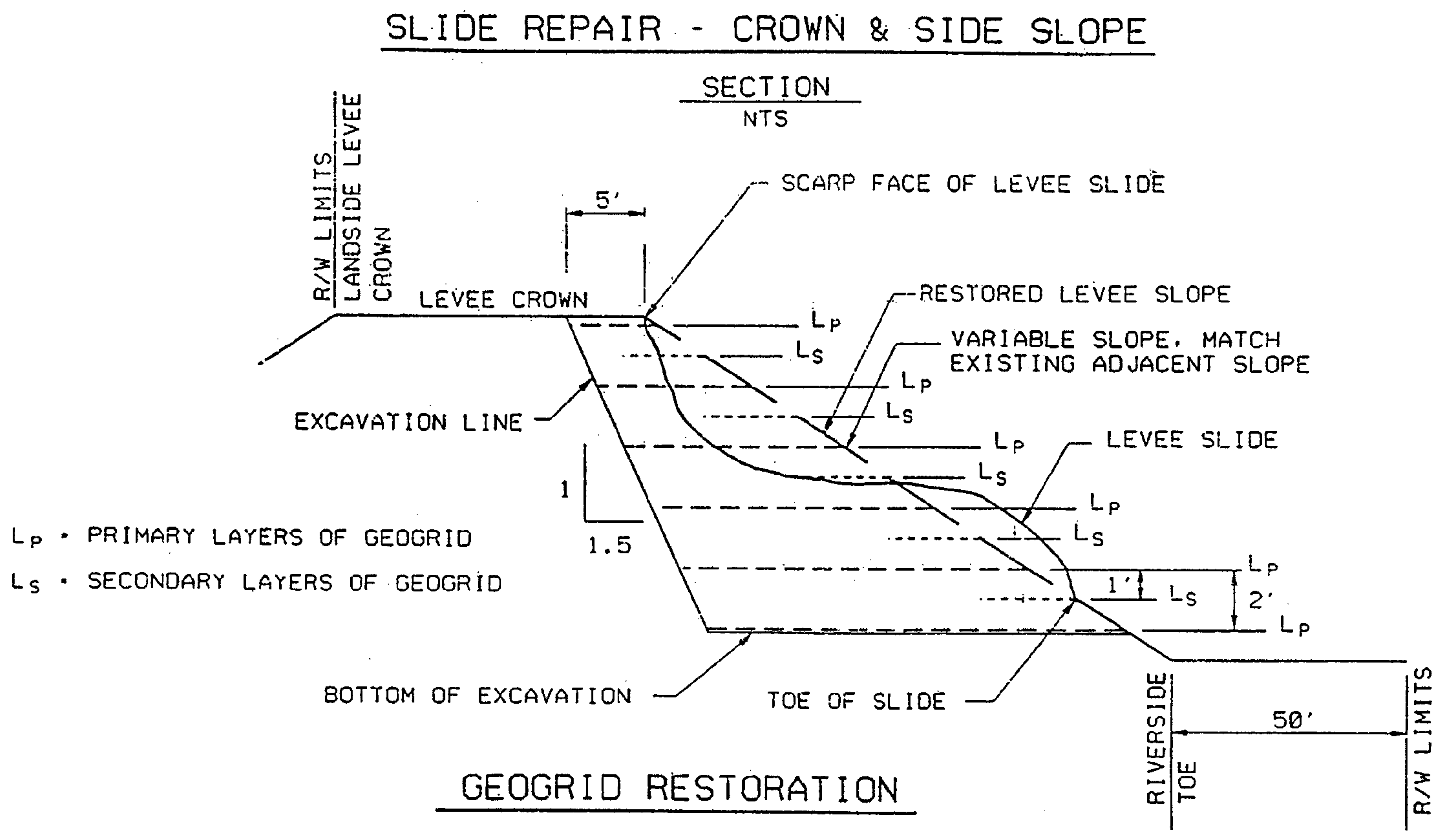


GEOGRID RESTORATION

NOTES:

1. PRIMARY LAYERS OF GEOGRID SHALL BE PLACED PERPENDICULAR TO THE LEVEE.
2. SECONDARY LAYERS OF GEOGRID SHALL BE PLACED PARALLEL TO THE LEVEE.
3. FOR CLARITY ONLY BOTTOM LAYER OF GEOGRID IS SHOWN.
4. PRIMARY AND SECONDARY LAYERS OF GEOGRID SHALL BE PLACED AT A MINIMUM OF 6 INCHES AND NOT TO EXCEED A MAXIMUM OF 12 INCHES FROM THE FACE OF THE SLOPE.

Figure 16. USAED, Memphis, slide repair using geogrids - plan view (Abernathy 1994)



NOTES:

1. EXCAVATION SHALL BEGIN 5 FT. BEHIND SCARP FACE AND EXTEND DOWNWARD ON A 1V ON 1.5H SLOPE TO AN ELEVATION 1.0 FT. BELOW THE TOE OF THE LEVEE SLIDE.
2. PRIMARY LAYERS OF GEOGRID SHALL EXTEND FROM THE FACE OF THE RESTORED LEVEE SLOPE TO THE EXCAVATION LINE.
3. SECONDARY LAYERS OF GEOGRID SHALL BE A MINIMUM OF 4.5 FT. IN WIDTH.
4. PRIMARY AND SECONDARY LAYERS OF GEOGRID SHALL BE PLACED AT A MINIMUM OF 6 INCHES AND NOT TO EXCEED A MAXIMUM OF 12 INCHES FROM THE FACE OF THE SLOPE.

Figure 17. USAED, Memphis, slide repair using geogrids - section view (Abernathy 1994)

Table 5
Overview of Innovative Rehabilitative Methods for Slope Instability

Rehabilitative Method	Applicable Conditions	Advantages	Disadvantages
Reinforced soil slope	Can be used with all soils used to rebuild levees	Allows construction to steeper slope (less expensive and useful when right-of-way is limited)	Steeper slopes (>1V:3H) are difficult to mow
	Slope can be rebuilt to any angle up to 90 deg	Does not require skilled labor or special equipment	Uncertainty of effect of construction damage and durability on tensile strength of polymeric reinforcement
Soil nailing	Applicable for levee slope with low factor of safety or experiencing limited creep	Relatively low cost	Steel nails may corrode in aggressive soil environments
	Not applicable for plastic clays with desiccation cracking or soft clays with significant creep	Uses light equipment	May not be feasible where underground utilities are present
		Rapid construction	Requires specialty contractor
		No excavation required (would create additional load on distressed slope)	
Pin piles	Applicable for levee slope with low factor of safety and competent underlying strata	Relatively low cost	Some movement required to mobilize support
		Uses light equipment	
		Rapid construction	
		No excavation required (would create additional load on distressed slope)	
Stone-fill trenches	Applicable for soils which remain stable during excavation with vertical side slopes to depth below failure surface	Relatively low cost	Stone must be available for backfill
		Rapid construction	Arching of soil between trenches not considered in design (conservative)
		Uses conventional readily available construction equipment	
		Provides drainage of slope when outlet provided	
Randomly distributed synthetic fibers	Not recommended with short smooth fibers	Mixes well in field	Short smooth fibers do not deter cracking when subjected to wet/dry cycles
		Zero cure time gives construction platform able to carry load	
		Insensitive to weather conditions (can compact wet of optimum)	
		Increase in shear strength allows for construction of steeper slopes	
		Normal rate of revegation (lime raises ph to 11-12 and deters revegation)	

(Continued)

Table 5 (Concluded)			
Rehabilitative Method	**Applicable Conditions**	**Advantages**	**Disadvantages**
Restraint structures	Applicable for small slides with competent underlying strata within 20 ft	Rapid construction	Construction cost increases rapidly with increased height of wall
	Clays with plasticity index greater than 30 and liquid limit greater than 50 require panels	Drilled shafts do not require excavation which would create additional load on distressed slope	
Geosynthetic drainage system	Applicable as temporary measure to improve surficial stability of plastic clays with desiccation cracking	Relatively low cost Automated method of installation	Possible smear of upper side wall during excavation of trench
			Possible piping of soil particles through the geotextile and into the drain
			Maintenance required to prevent possible blockage of outlets by silt accumulation and/or grass root intrusion
Lime-fly ash injection	Applicable for rehabilitation or prevention of slope failure for plastic clays with desiccation cracking	Relatively low cost	Long-term performance needs to be documented
		Uses light equipment	
		No excavation required (would create additional load on distressed slope)	
Anchored geosynthetic system	Best suited for sandy slopes with shallow ($\leq$ 10 ft) failure surface	Relatively low cost	Restricts activities (mowing, livestock) on levee until vegetation is well established
		Uses light equipment	
		Rapid construction	
		Does not require skilled labor or special equipment	
		Well-suited for environmentally sensitive areas (physically intrusive, erosion resistant, and promotes establishment of vegetation)	

For purposes of definition, rehabilitation of levee slopes would involve steep slopes (angle less than 70 deg or 2.75V:1H) on hard foundations (foundation does not influence design of slope) and permanent structures (expected life greater than 5 years - typically 75 to 100 years for levees) as discussed by Holtz, Christopher, and Berg (1995). General guidance on using geotextiles to build reinforced soil walls is given in TM 5-818-8 (HQUSACE 1995b). Guidance on the use of geotextiles and geogrids to construct reinforced soil slopes is given in an Engineer Manual entitled "Slope Stability" (HQUSACE in preparation) and in a report by Duncan, Sehn, and Bosco (1988). At the present time, the definitive information on design of RSS is given in reports from the Federal Highway Administration Demonstration Project 82 (Elias and Christopher 1997, Elias 1997).

The factors of safety for reinforced slopes are the same as those for slopes without reinforcement (HQUSACE 1978, HQUSACE in preparation):

a. End-of-construction, riverside and landside, FS $\geq$ 1.3.

b. Steady seepage from full river stage (long-term stability), riverside, FS $\geq$ 1.5.

c. Sudden drawdown, riverside, FS $\geq$ 1.0.

There are three failure modes for RSS:

a. Internal, with the failure plane passing through the reinforcement.

b. External, with the failure surface passing behind and underneath the reinforced mass.

c. Compound, with the failure plane passing behind and through the reinforced mass.

The design of RSS must consider internal stability (pullout and tensile failure of reinforcement), external stability (deep-seated overall instability, bearing-capacity failure, excessive settlement and sliding instability), face stability (intermediate reinforcement), and seismic analysis (if applicable).

As previously mentioned, some uncertainty exists regarding the effect of construction installation damage and durability (chemical and biological degradation with time) on the tensile strength of polymeric reinforcements. Because of varying polymer types, quality and additives, each is different in its resistance to aging and attack by different chemical and biological agents and must be investigated individually. Polyester products are susceptible to aging strength reduction due to hydrolysis (water availability) and high temperatures, while polypropylene products are susceptible to aging strength losses due to oxidation (contact with oxygen) and/or high temperatures. In addition to construction installation damage and durability effects on the tensile strength of polymeric reinforcements, creep (deformation under sustained load) may also occur. Currently creep tests are carried out in air (Elias and Christopher 1997). As mentioned in Chapter 2, recent laboratory experiments with a soil-geosynthetic composite indicate that geosynthetic creep does not occur with granular backfill (Ketchart and Wu 1996).

The allowable tensile strength of the geosynthetic reinforcement which considers strength losses over the design life period of the RSS is as follows (Elias and Christopher 1997, Elias 1997) (multiplying the reduction factors together is conservative (Ingold 1992; Berg, Allen, and Bell 1998)):

$$T_{al} = \frac{T_{ULT}}{RF_{ID} \times RF_{CR} \times RF_{D}}$$

where

T_{al} = Long-term tensile strength as load per unit width of reinforcing

T_{ULT} = Ultimate (or yield) tensile strength from a wide strength tensile strength test

RF_{ID} = Installation damage reduction factor. Not time dependent since it occurs during backfill placement and compaction. Depends on the weight and type of construction equipment used, gradation and angularity of backfill, lift thickness, weight and type of geosynthetic. Best determined by full-scale experiments. If full-scale data are not available, the installation damage reduction factor may be estimated using Table 6 (Elias 1997).

RF_{CR} = Creep reduction factor. Ratio of the ultimate strength (T_{ULT}) to the creep limit strength obtained from laboratory creep tests on multiple product samples loaded to various percentages of the ultimate load for periods of up to 10,000 hr. Typical ranges of creep reduction factors as a function of polymer type are shown below. As previously discussed, recent laboratory tests with a soil-geosynthetic composite indicate that geosynthetic creep does not occur with granular backfill (Ketchart and Wu 1996). Therefore, these creep reduction factors may be highly conservative when granular backfill is used.

Polymer Type	Creep Reduction Factor
Polyester	2.5 to 2.0
Polypropylene	5.0 to 4.0
Polyethylene	5.0 to 2.5

RF_D = Durability reduction factor. Depends on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis, and stress cracking and typically varies from 1.1 to 2.0.

Soil backfill requirements for RSS are (Elias and Christopher 1997):

Sieve Size	Percent Passing
20 mm[1]	100 - 75
No. 4 (4.75 mm)	100 - 20
No. 40 (0.425 mm)	0 - 60
No. 200 (0.075 mm)	0 - 50

[1] The maximum fill particle size can be increased (up to 100 mm) provided field tests are performed to evaluate potential tensile strength reduction due to construction damage.

Table 6 Installation Damage Reduction Factors Elias (1997)		
Geosynthetic	**Type 1 Backfill Max. Size 102 mm D_{50} about 30 mm**	**Type 2 Backfill Max. Size 20 mm D_{50} about 0.7 mm**
High-density polyethylene unaxial geogrid	1.20 - 1.45	1.10 - 1.20
Polypropylene biaxial geogrid	1.20 - 1.45	1.10 - 1.20
PVC coated polyester geogrid	1.30 - 1.85	1.10 - 1.30
Acrylic coated polyester geogrid	1.30 - 2.05	1.20 - 1.40
Woven geotextiles (polypropylene and polyester)[1]	1.40 - 2.20	1.10 - 1.40
Nonwoven geotextiles (polypropylene and polyester)[1]	1.40 - 2.50	1.10 - 1.40
Slit film woven polypropylene geotextiles[1]	1.60 - 3.00	1.10 - 2.00

[1] Minimum weight of geotextile is 270 g/m^2.

As a general rule, the soil backfill plasticity index should not exceed 20. However, the conventional plasticity-index criteria may have to be tempered with local experience to avoid rejecting suitable soils, especially if low-plasticity soils are not locally prevalent. Magnesium sulfate soundness loss is less than 30 percent after four cycles, or equivalent sodium sulfate soundness loss is less than 15 percent after three cycles according to AASHTO T-104 (AASHTO 1996).

As discussed in Chapter 2, research is ongoing concerning use of poorly draining soils, such as clays and silts, as backfill (Chou and Wu 1993; Sabatini et al. 1997). Clayey backfills have poor drainage and the potential to undergo creep deformation. Despite these problems, clayey soil may be used as backfill material if suitable design procedures are followed (AASHTO 1996, Sabatini et al. 1997). Geotextile reinforcements (primary and secondary layers) must be more permeable (in the transverse direction) than the fill material to prevent accumulation of water above the geotextile during seepage and/or infiltration of rainwater (Elias and Christopher 1997). Using permeable geotextiles will dissipate excess pore water pressures and increase the soil shear resistance along the potential failure plane and pullout resistance along the soil-geotextile interface. Nonwoven geotextiles offer high in-plane transmissivity and have excellent interface contact properties. However, nonwoven geotextiles have relatively low tensile strength, an important parameter in the design of geotextile reinforced structures. Therefore, a woven/nonwoven composite geotextile or high-strength nonwoven geotextile would be appropriate for reinforcement of poorly draining fills. The design of using geotextiles as reinforcements of poorly draining fills requires determination of the mechanical and hydraulic characteristics of the geotextile. In situ conditions including changes with time must be considered when making this determination. Confinement may increase the stiffness and strength of the geotextile but decrease the in-plane transmissivity.

This information is needed to determine the spacing of geotextiles for dissipation of excess pore water pressures and stability against reinforcement breakage and pullout (Zornberg and Mitchell 1992; Cuenca and Wright 1988; Department of Transportation 1994; Zornberg et al. 1995).

Design charts for RSS can be used for preliminary evaluation of internal stability and to check the results of computer analysis (Christopher and Leshchinsky 1991; Holtz, Christopher, and Berg 1995; Jewell 1990b; Schmertmann; Yamanouchi and Fukuda 1993; Elias and Christopher 1997; Abramson et al. 1993, 1996). Charts by Jewell, (Figures 18 to 20), which consider pore water pressure (Figure 21), are useful (Jewell 1990a, Lawson 1992, Department of Transportation 1994). Also, charts, based on a pseudostatic limit equilibrium analysis which consider horizontal acceleration and incorporate a permanent displacement limit, are available for seismic design of RSS (Ling, Leshchinsky, and Perry 1997).

Computer programs for the design of reinforced slopes are available from companies which market soil reinforcement (Tensor Corporation 1994, Strata Systems Inc. 1993). These programs are generally used to evaluate the reinforcement layout and in some cases are reinforcement-specific (Elias and Christopher 1997). Two generic programs are available for both reinforcement design and evaluation of reinforcement layout:

The computer program ReSlope, developed under this study (REMR Work Unit 32646, Levee Rehabilitation), yields the optimal length and spacing of the geosynthetic given user-specified safety factors and geosynthetic ultimate strength. ReSlope provides recommendations regarding selection of soil shear strength parameters, safety factors, and reinforcement layout. The program performs a conventional slope stability analysis against deep-seated failure (Leshchinsky 1994, 1997a,b).

The computer program RSS, developed for the Federal Highway Administration, can determine the required spacing of a user-specified geosynthetic to achieve a specified safety factor, the required geosynthetic ultimate strength for a given spacing to achieve a specified safety factor, and the factor of safety for a specified pattern of reinforcement. RSS provides recommendations regarding selection of soil shear strength parameters, safety factors, and reinforcement layout. The program evaluates external stability by preforming conventional slope stability analysis on an unreinforced slope (Marr and Werden 1997).

Unless the reinforcement is wrapped around at the slope face, face stability must be considered in the design of a reinforced slope. Neither ReSlope or RSS analyze face stability (RSS indicates intermediate reinforcement is needed if the computed vertical spacing between primary reinforcement layers exceeds 2 ft). For slopes flatter than 1H:1V (most levee slopes would fall in this category), with the vertical spacing between primary reinforcement no greater than 1.3 ft and minimum seepage and sloughing occurring at the face of the slope (slope

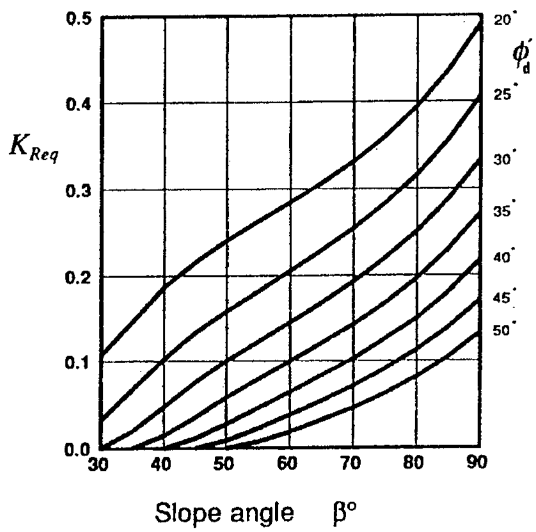
STEEP REINFORCED SLOPE DESIGN CHARTS

Jewell (1990)

CHART 1

$$r_u = \frac{u}{\gamma z} = 0.00$$

Minimum Required Force K_{Req}



Minimum reinforcement length:

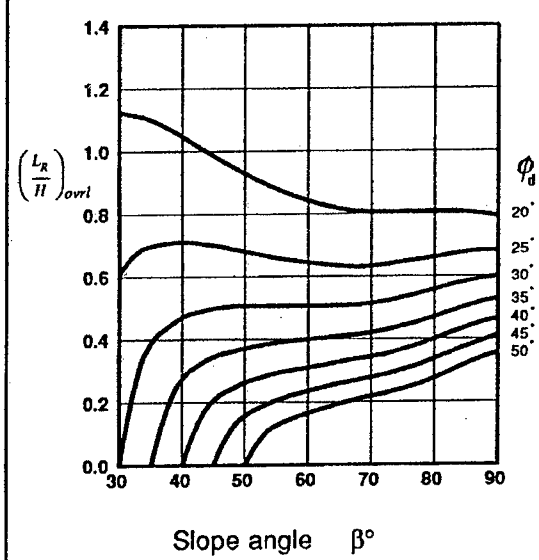
(1) The minimum length at the crest of the slope is that required for *overall stability*.

(2) The minimum length at the base of the slope is the greater of that required for *overall stability* and to prevent *direct sliding*.

(3) Where reinforcement of constant length is to be used select the greater length required to satisfy equilibrium at the base of the slope, (2) above.

(4) Where *direct sliding* governs the required reinforcement length at the base of the slope it is permissible to reduce the length uniformly from L_{ds} at the base of the slope to L_{ovrl} at the crest of the slope.

Minimum Required Length
Overall Stability $(L_R/H)_{ovrl}$



Minimum Required Length
Direct Sliding $(L_R/H)_{ds}$

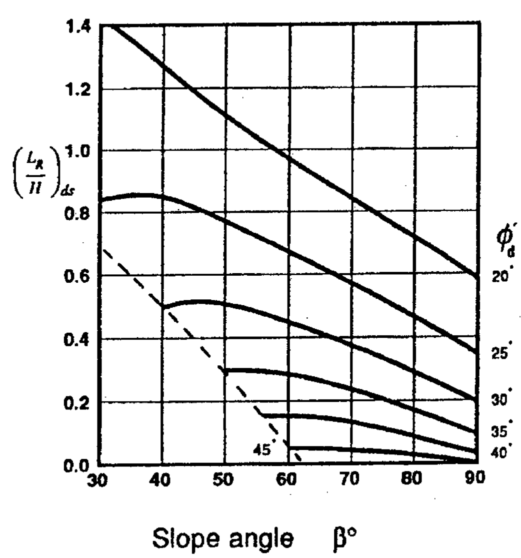
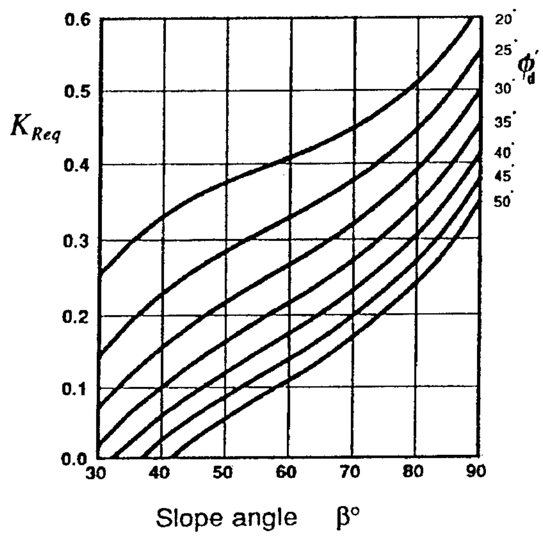


Figure 18. Steep reinforced slope design chart, r_u = 0.00 (courtesy of Jewell 1990a)

STEEP REINFORCED SLOPE DESIGN CHARTS Jewell (1990)	CHART 2 $r_u = \frac{u}{\gamma z} = 0.25$

Minimum Required Force K_{Req}



Minimum reinforcement length:

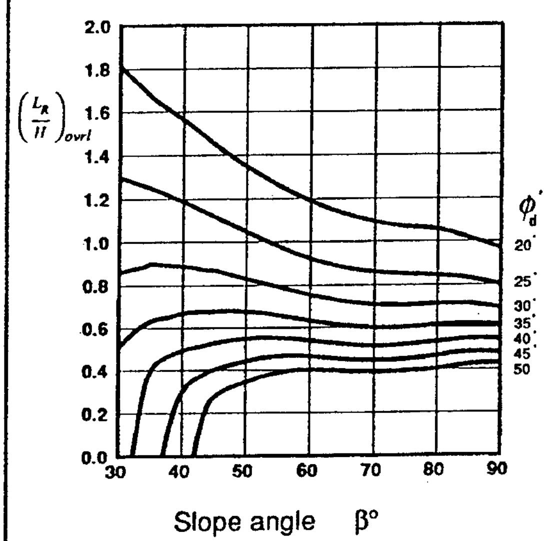
(1) The minimum length at the crest of the slope is that required for *overall stability.*

(2) The minimum length at the base of the slope is the greater of that required for *overall stability* and to prevent *direct sliding*.

(3) Where reinforcement of constant length is to be used select the greater length required to satisfy equilibrium at the base of the slope, (2) above.

(4) Where *direct sliding* governs the required reinforcement length at the base of the slope it is permissible to reduce the length uniformly from L_{ds} at the base of the slope to L_{ovrl} at the crest of the slope.

Minimum Required Length
Overall Stability $(L_R/H)_{ovrl}$



Minimum Required Length
Direct Sliding $(L_R/H)_{ds}$

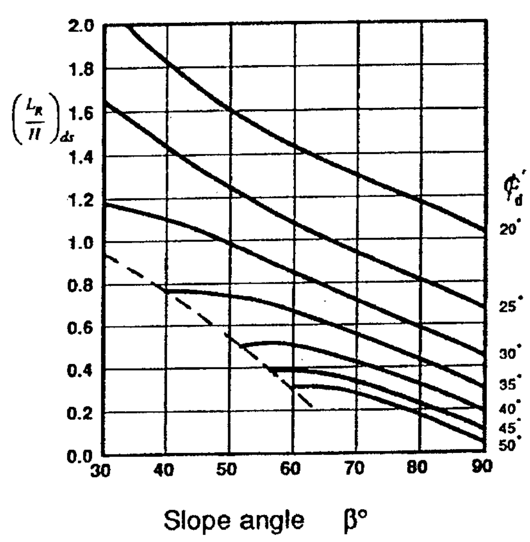
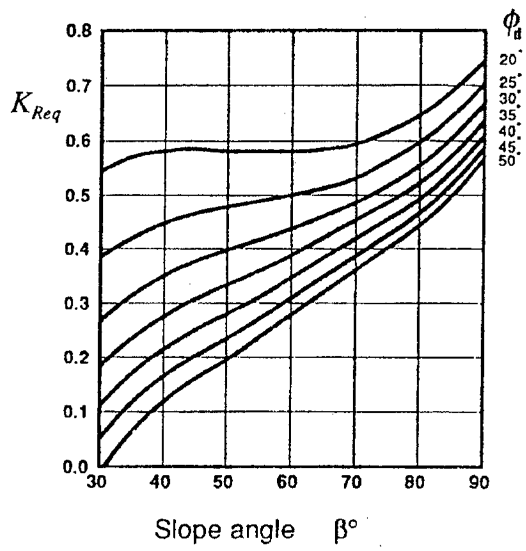


Figure 19. Steep reinforced slope design chart, $r_u = 0.25$ (courtesy of Jewell 1990a)

STEEP REINFORCED SLOPE DESIGN CHARTS Jewell (1990)	CHART 3 $r_u = \frac{u}{\gamma z} = 0.50$

Minimum Required Force K_{Req}



Minimum reinforcement length:

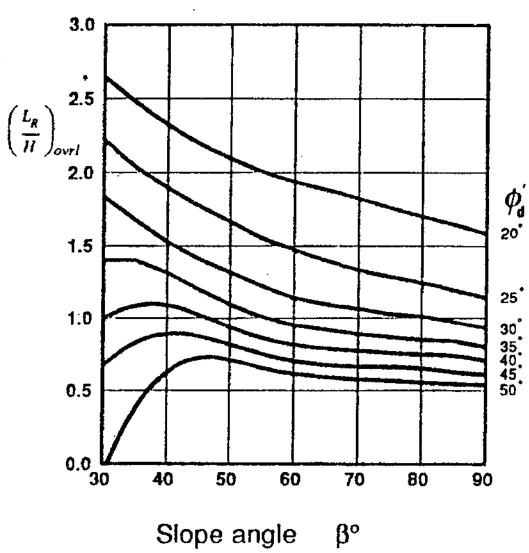
(1) The minimum length at the crest of the slope is that required for *overall stability*.

(2) The minimum length at the base of the slope is the greater of that required for *overall stability* and to prevent *direct sliding*.

(3) Where reinforcement of constant length is to be used select the greater length required to satisfy equilibrium at the base of the slope, (2) above.

(4) Where *direct sliding* governs the required reinforcement length at the base of the slope it is permissible to reduce the length uniformly from L_{ds} at the base of the slope to L_{ovrl} at the crest of the slope.

Minimum Required Length
Overall Stability $(L_R/H)_{ovrl}$



Minimum Required Length
Direct Sliding $(L_R/H)_{ds}$

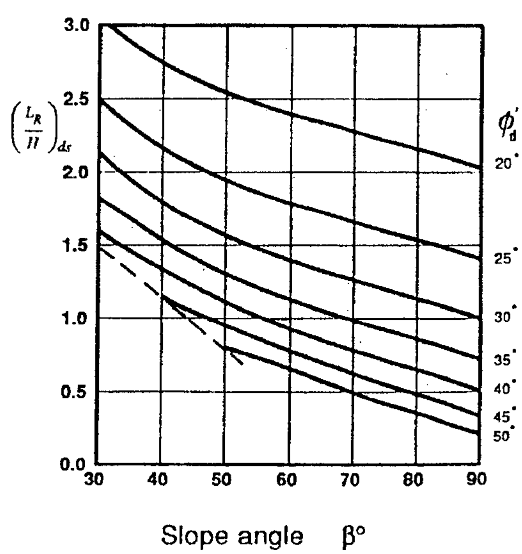


Figure 20. Steep reinforced slope design chart, $r_u = 0.50$ (courtesy of Jewell 1990a)

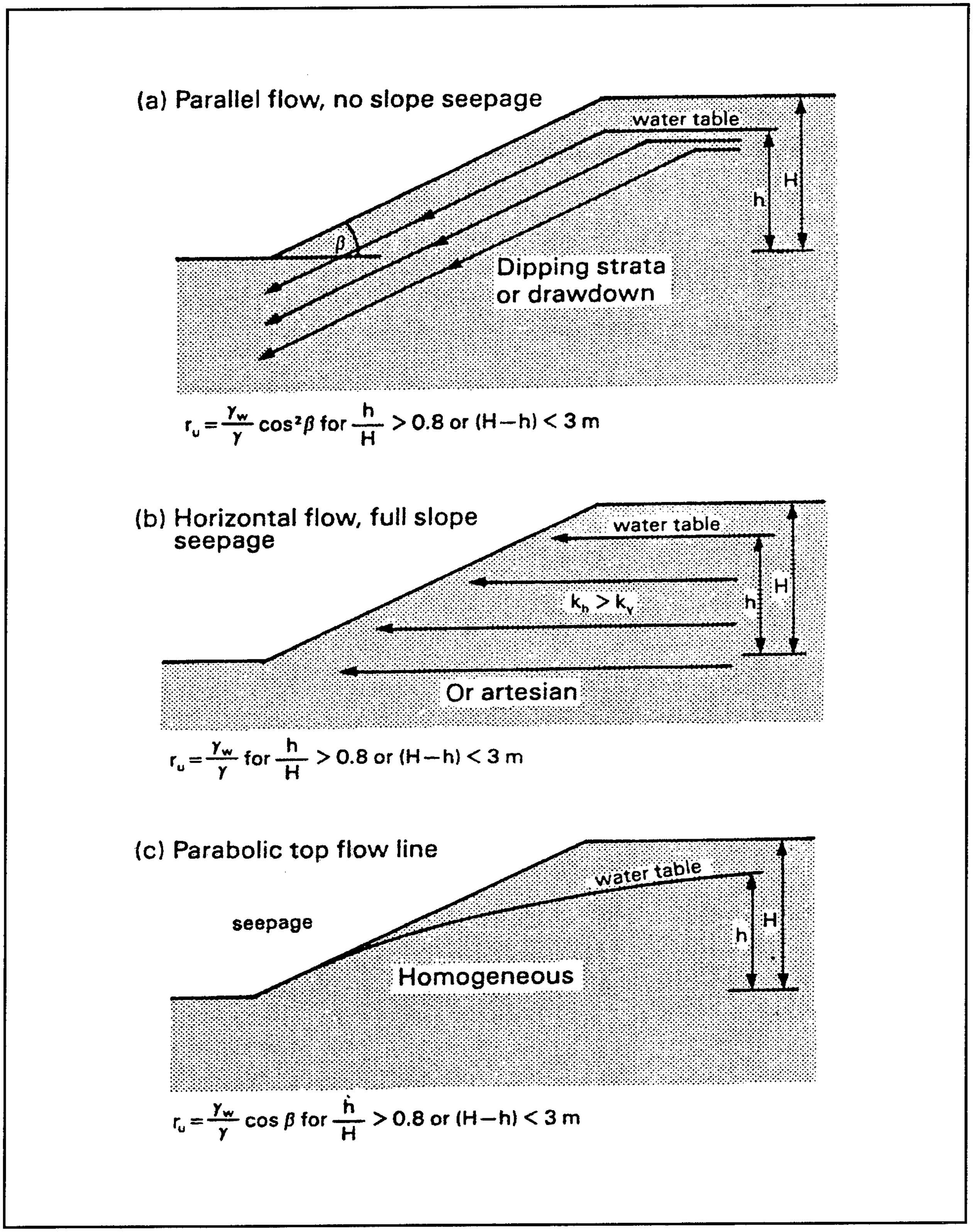


Figure 21. Values of the pore water pressure parameter r_u for various flow conditions (Crown copyright is reproduced with the permission of the Controller of Her Majesty's Stationery Office, Department of Transportation 1994)

angle is greater than angle of internal friction of the soil), it would probably not be necessary to wrap the face (Holtz, Christopher, and Berg 1995). If the face is not wrapped, a face stability analysis (Figure 22) should be performed using the method of Thielen and Collin (1993) to determine the frequency, vertical spacing, strength, and length of intermediate reinforcement (Collin 1996, Simac 1992). In addition to increasing face stability, intermediate reinforcement assists in obtaining good compaction near the slope face, especially in cohesionless soils, by allowing compaction equipment to operate effectively near the edge of the slope (Koerner and Wilson-Fahmy 1991, Simac 1992). For steep (> 50 deg) slopes, temporary support near the slope face, as shown in Figure 23, may be needed to make construction feasible (Leshchinsky 1997a,b).

Erosion control and revegetation are an integral part of slope rehabilitation. Slope facing requirements will depend on the reinforcement spacing, slope angle, and soil type as shown in Table 7. Erosion control measures may range from temporary to permanent-armored systems (Collin 1996, Elias and Christopher 1997).

In summary, soil reinforcement using geotextiles or geogrids offer an excellent method for rehabilitation of levee slopes. RSS can utilize existing levee soils and rebuild to steeper slopes with resulting cost savings and (if needed) utilization of less right-of-way. Although some uncertainty exists regarding the effect of durability on the tensile strength of polymeric reinforcements, adequate design methods are available for obtaining safe and cost-competitive rehabilitation of levee slopes.

Soil Nailing

Another innovative method to correct slides on levees is soil nailing. Soil nailing consists of inserting steel rods or "nails" into soil to stabilize the soil mass. The nails can be driven or placed in tremie grouted predrilled boreholes. The nails are not posttensioned as tiebacks are. Soil nailing has been used to construct excavations and stabilize slopes for approximately 25 years. It is applicable where little or no movement is occurring but where safety factors indicate future movement may occur or creeping slopes in which movement is occurring. Soil nailing is not applicable for plastic clays with desiccation cracking or soft clays with significant creep (requires rigid piles or piers with significant bending capacity installed near the toe of the slide, i.e. a restraint structure - discussed below). The sliding mass is usually uniformly reinforced by relatively closely spaced nails as shown in Figure 24 (Mitchell and Villet 1987; Elias and Juran 1991; Walkinshaw and Chassie 1994; Porterfield, Cotton, and Byrne 1994; Holtz and Schuster 1996; Byrne et al. 1996).

The USAED, New Orleans, considered soil nailing to stabilize a 700-ft section of the Mississippi River bank in Baton Rouge, LA, which was experiencing a creep failure into the river (Satterlee 1994). The USAED, Vicksburg, is considering using soil nailing to stabilize a section of the Mississippi River bank in Natchez, MS.

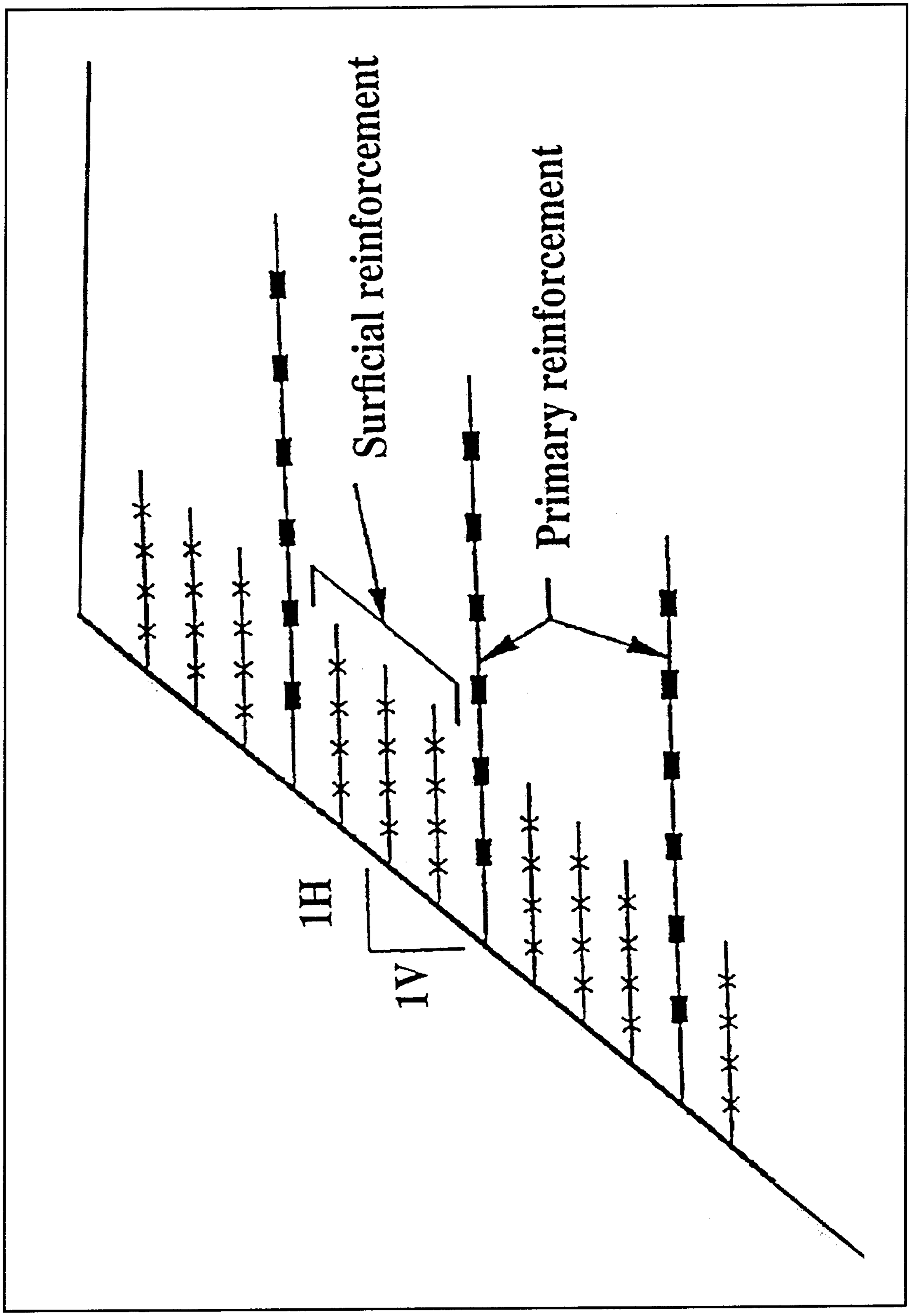


Figure 22. Cross section of reinforced soil slope (adapted from Collin 1996)

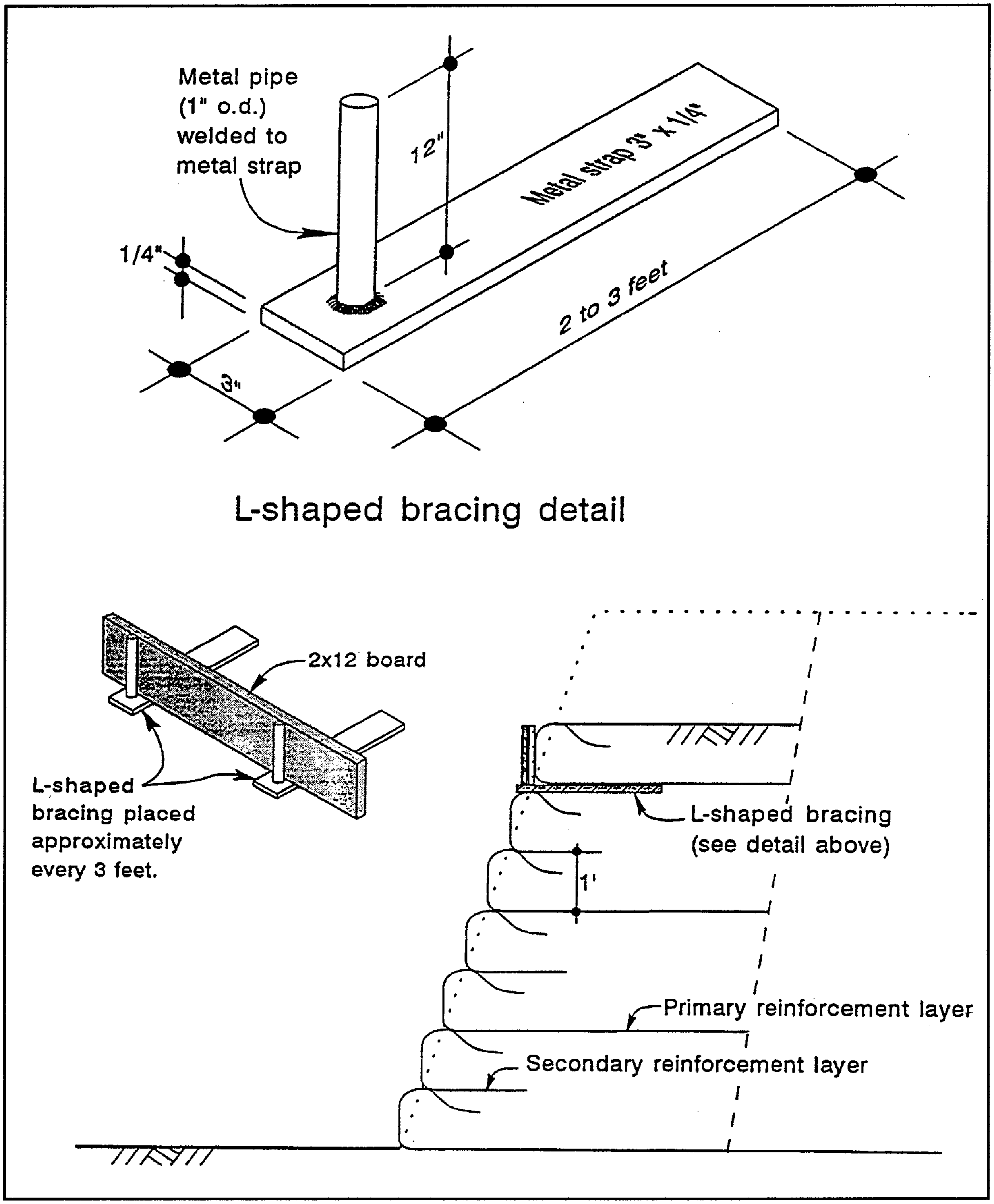


Figure 23. Removable face support used in construction of steep reinforced soil slope (from Leshchinsky 1997b)

Table 7
RSS Slope Facing Options (after Collin 1996, Elias and Christopher 1997)

Slope Face Angle Soil Type	Type of Facing			
	Geosynthetic not Wrapped at Face		Geosynthetic Wrapped at Face	
	Vegetated Face	Hard Facing	Vegetated Face	Hard Facing
> 50 deg All soil types	Not recommended	Gabions	Sod permanent erosion blanket with seed	Wire baskets Stone Shotcrete
35 to 50 deg Clean sands (SP) Rounded gravel (GP)	Not recommended	Gabions Soil Cement	Sod permanent erosion blanket with seed	Wire baskets Stone Shotcrete
35 to 50 deg silts (ML) Sandy silts (ML)	Bioreinforcement Drainage Composites	Gabions Soil Cement Stone Veneer	Sod permanent erosion blanket with seed	Wire baskets Stone Shotcrete
35 to 50 deg Silty sands (SM) Clayey sands (SC) Well-graded sands and Gravels (SW and GW)	Temporary erosion blanket with seed or sod Permanent erosion mat with seed or sod	Hard facing not needed	Geosynthetic wrap not needed	Geosynthetic wrap not needed
25 to 35 deg All soil types	Temporary erosion blanket with seed or sod Permanent erosion mat with seed or sod	Hard facing not needed	Geosynthetic wrap not needed	Geosynthetic wrap not needed

Advantages and disadvantages of soil nailing are given in Table 5 (Schlosser 1993; Abramson et al. 1993; Byrne et al. 1993; Ortigao, Palmeira, and Zirlis 1995). The advantages of soil nailing are low cost, light equipment, and rapid construction with no excavation required that would create additional load on a distressed slope. The disadvantages of soil nailing are that steel nails may corrode in aggressive soil environments, may not be feasible where underground utilities are present, and must be metalled by specialty contractors.

The design of soil nailing is given in the report from the Federal Highway Administration Demonstration Project 103 (Byrne et al. 1996; Sabatini et al. 1997). The computer program SNAIL, developed by the California Department of Transportation, determines the minimum factor of safety for a one- or two-layer soil system including provisions for surcharge and groundwater (California Department of Transportation 1996). The computer program GoldNail, developed by Golder Associates, can handle multilayered soil profiles and design a soil nail pattern that satisfies specified conditions (factors of safety or load resistance factors, soil conditions, and nail strength), calculate the factor of safety for a given nail pattern, or estimate the nail service loads rather than identifying the required nail tensile strength (Golder Associates 1996).

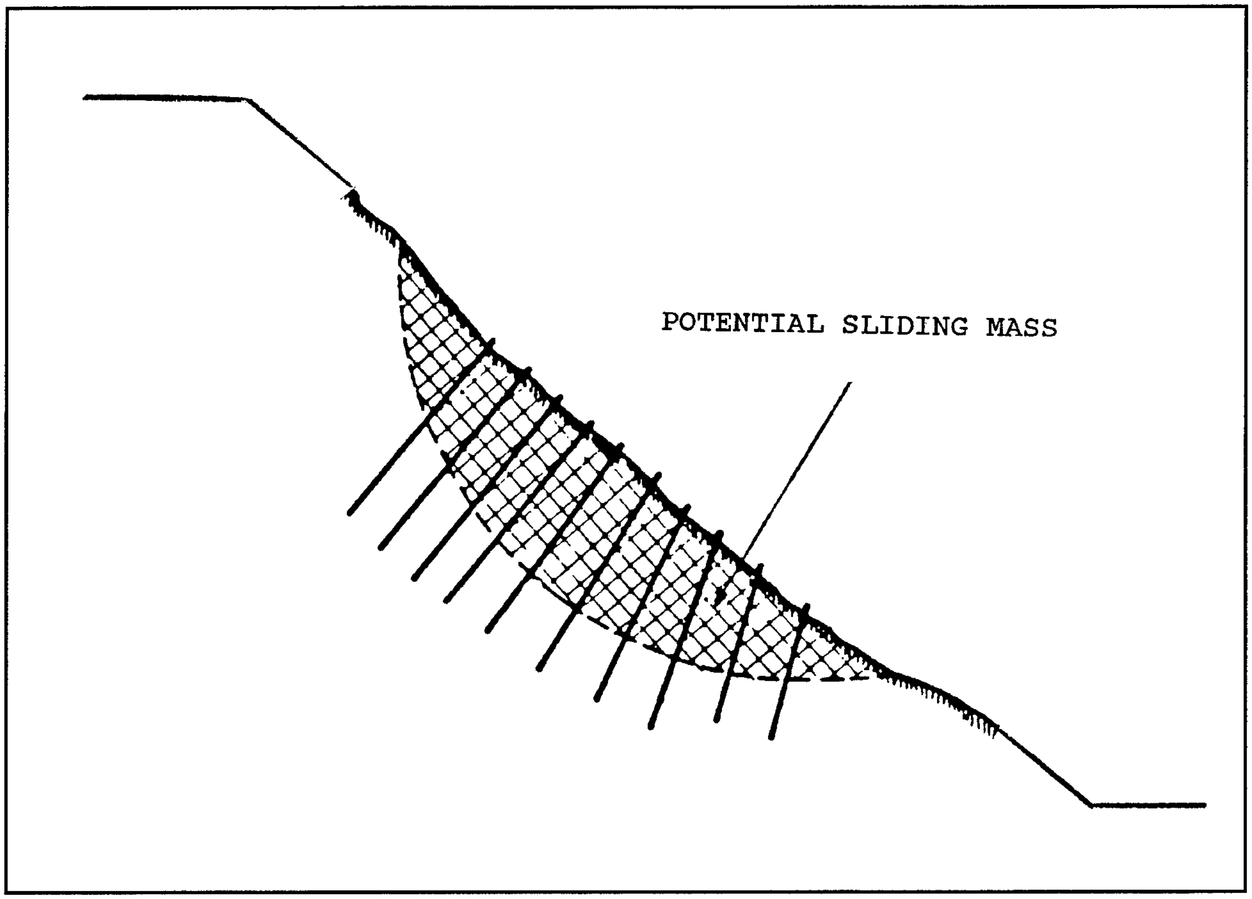


Figure 24. Use of soil nailing to stabilize levee slope

In summary, soil nailing offers an excellent method for strengthening levee slopes. It is not applicable for plastic clays with desiccation cracking or soft clays with significant creep. It offers low cost, use of light equipment, and rapid construction, with no excavation required that would create additional load on a distressed slope. Consideration must be given to corrosion in aggressive soil environments. Soil nailing may not be feasible where underground utilities are present. A specialty contractor is required to install soil nailing.

Pin Piles

Pin piles are a viable option to stabilize a slope where little or no movement is occurring but where slope stability analyses indicate future movement may occur and where there is a competent underlying strata (lower part of the levee or foundation) that the potentially unstable soil mass can be anchored to (Sabatini et al. 1997). The definition of competent underlying strata would depend on the levee soil and foundation conditions at the location. For a levee constructed of montmorillonitic clay, competent underlying strata would be below the depth of desiccation, while for a levee constructed of silt, competent underlying strata might be a clay layer in the foundation. Attempts to stabilize

embankments with piles which were not tied into a competent underlying strata have not been successful (Burns et al. 1990).

Reticulated root piles were developed in the 1950's by Lizzi and patented by the firm Fondedile of Naples, Italy (Holtz and Schuster 1996, Lizzi 1978). The USAEWES participated in the instrumentation and monitoring of early root pile installations in the United States (Palmerton 1984).

Pin piles, also known as micropiles, or root piles or dowels, are cast-in-place reinforced concrete piles with diameters from 3 to 12 in. Small-diameter pin piles have a reinforcing rod or steel pipe in the center, while large-diameter pin piles may utilize a reinforcing bar cage with spiral reinforcement. A pin pile system forms a block of reinforced soil that extends below the critical failure surface into a competent underlying strata as shown in Figure 25. Pin piles may be installed in a reticulated (crisscrossing) pattern (root piles) or in a combination of vertical and batter orientations (Insert Walls^{SM}) spaced further apart than for pin piles. The group effect of the reticulated pin pile system provides greater shear capacity than would closely spaced vertical piles (Lizzi 1978; Ting et al. 1990; Pearlman, Campbell, and Withiam 1992; Holtz and Schuster 1996; Abramson et al. 1996; Sabatini et al. 1997; Munfakh 1997).

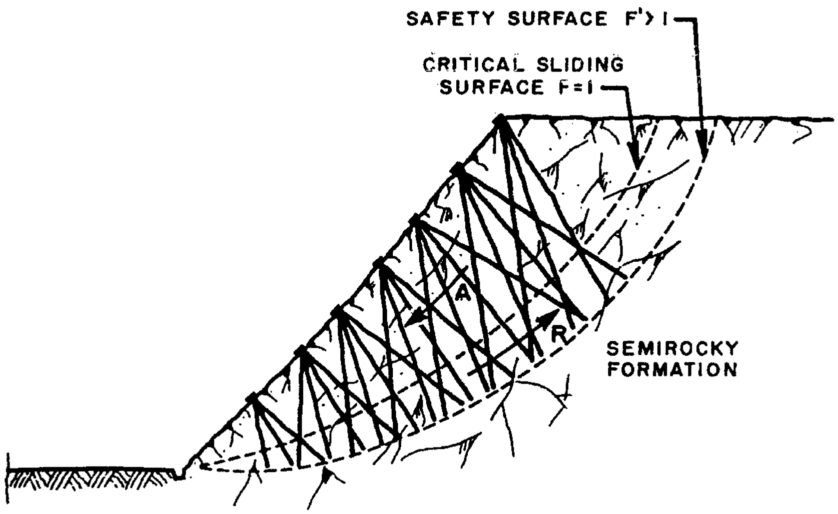
Advantages and disadvantages of the pin pile system are given in Table 5. The advantages of pin piles are low cost, light equipment, and rapid construction with no excavation required that would create additional load on a distressed slope. (Abramson et al. 1996; Sabatini et al. 1997). The disadvantage is that some movement may be required to mobilize support (Dash 1987).

The design of pin piles assumes the pin piles and retained soil between the pin piles act as a composite mass to resist lateral earth pressures. The pin piles extend below the critical failure surface and anchor the potentially unstable soil mass to the competent underlying strata. The shear stresses on any potential failure surface through the composite mass are resisted by the allowable shear stresses of the pin pile soil combination with a prescribed factor of safety (Datye and Nagaraju 1980; Ito, Matsui, and Hong 1982; Winter, Schwarz, and Gudehus 1983; Wichter, Krauter, and Meiniger 1988; Allison, Mawditt, and Williams 1991; Sabatini et al. 1997).

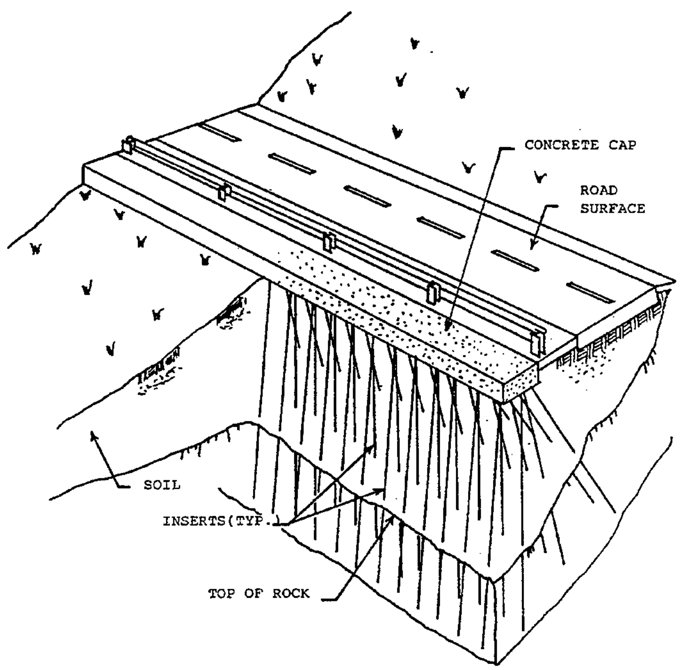
Insert Walls^{SM} are also designed to resist lateral earth pressures. Performance data indicate that wall movements are typically concentrated along a localized plane and that the walls are relatively flexible and do not behave as rigid gravity walls. The design for Insert Walls^{SM} includes the following steps (Pearlman, Campbell, and Withiam 1992; Sabatini et al. 1997; Abramson et al. 1996):

a. Conduct stability analyses to determine the increase in resistance along a potential or existing failure surface required to provide an adequate factor of safety.

b. Check the potential for structural failure of the pin piles due to loading from the moving soil mass using Figure 26.



a. Reticulated (crisscrossing) pattern (root piles) (courtesy of American Society of Civil Engineers, Lizzi 1978)



b. Combination of vertical and batter orientations (Insert Walls℠) (courtesy of Dash 1987)

Figure 25. Pin pile system to stabilize slope

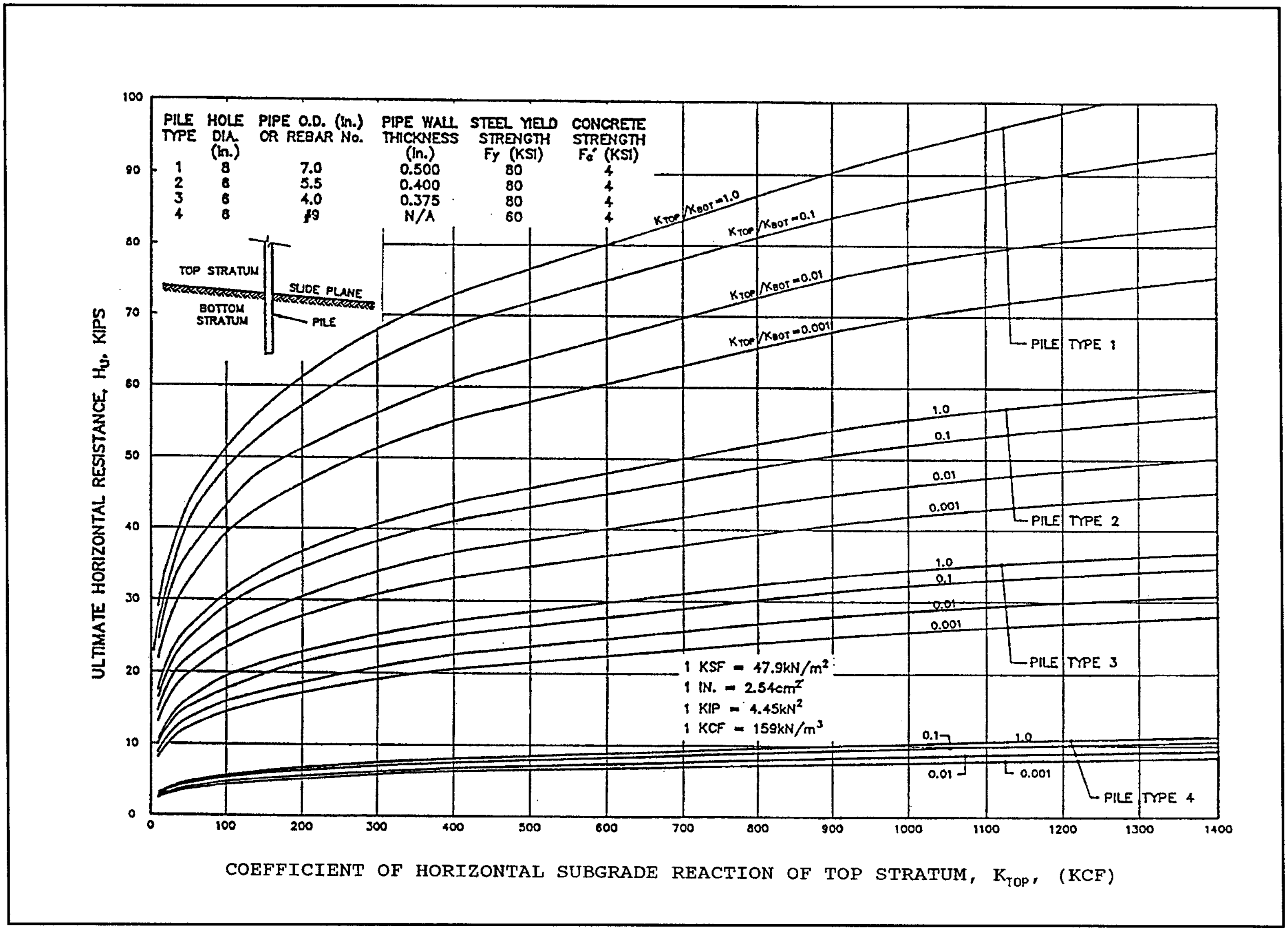


Figure 26. Preliminary design chart for ultimate horizontal resistance of pin piles (courtesy of ASCE, Pearlman, Campbell, and Withiam 1992)

c. Check the potential for plastic flow of soil around the pin piles as shown in Figure 27. This type of failure may occur if the soil above the failure surface is relatively soft and the pin piles are stiff and spaced far apart.

Additional information on the design, construction, and performance of pin walls is given in reports from the Federal Highway Administration Demonstration (Bruce and Juran 1997a,b,c,d).

The construction sequence for Insert Walls^{SM} is shown in Figure 28. A steel reinforced concrete cap beam is constructed at the ground surface. Corrugated polyethylene sleeves are placed in the formwork for the cap beam at locations and inclinations for each pin pile prior to concrete placement. Rotary drilling equipment is aligned in the sleeves, and holes are drilled for the pin piles. The hole is tremie grouted, and a reinforcing rod or steel pipe is inserted and pressure grouted in place as the casing is removed (Sabatini et al. 1997; Dash 1987).

In summary, pin piles are a viable option to stabilize a slope where little or no movement is occurring but where slope stability analyses indicate future movement may occur and where there is a competent underlying strata (lower part of the levee or foundation) that the potentially unstable soil mass can be anchored to. It is not applicable for plastic clays with desiccation cracking or soft clays with significant creep. It offers low cost, use of light equipment, rapid construction, and does not require excavation that would create additional load on a distressed slope. Some movement may be required to mobilize support.

Stone-Fill Trenches

Stone-fill trenches consist of trenches excavated below the failure surface and partially filled with stone as shown in Figures 29 and 30. The method is applicable for soils which will remain stable where trenches are excavated with vertical side slopes to a depth below the failure surface. Stone-fill trenches have been used by the USAED, Vicksburg, to stabilize slopes since 1982 (Wardlaw et al. 1984; Sills and Fleming 1992, 1994; Longmire 1992a,b).

In 1982, a shallow translatory slide in a plastic clay on the Big Sunflower River near Clarksdale, MS, was stabilized using 14 trenches excavated perpendicular to the slope to a 3-ft depth below the failure surface and filled to within 3 ft of the ground surface with a commercially available washed gravel aggregate. It was felt that placing the trenches perpendicular to the slope and by providing an outlet on the downstream end, drainage would reduce the piezometric pressure on the failure surface and increase the stability (Wardlaw et al. 1984). Subsequent movement of the slope indicated that (angular) stone, not (smooth) gravel, should be used as backfill. Other applications by the USAED, Vicksburg, include use of stone-fill trenches to stabilize slides on the inlet channel at John H. Overton Lock and Dam on the Red River Waterway and on the Ouachita River at Rilla, LA (Sills and Fleming 1992, 1994).

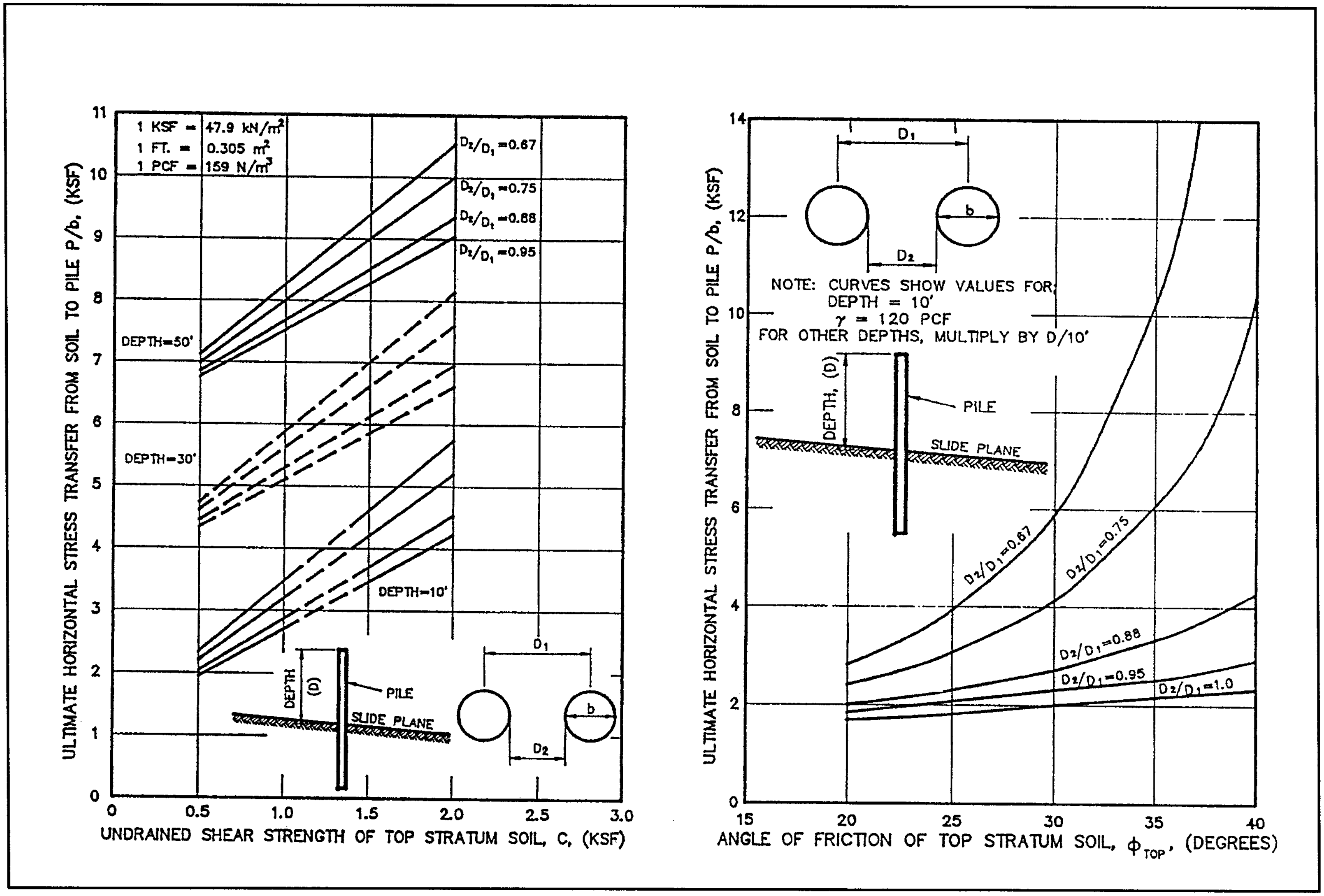


Figure 27. Ultimate stress transfer from soil to pine pile versus shear strength of soil (courtesy of ASCE, Pearlman, Campbell, and Withiam 1992)

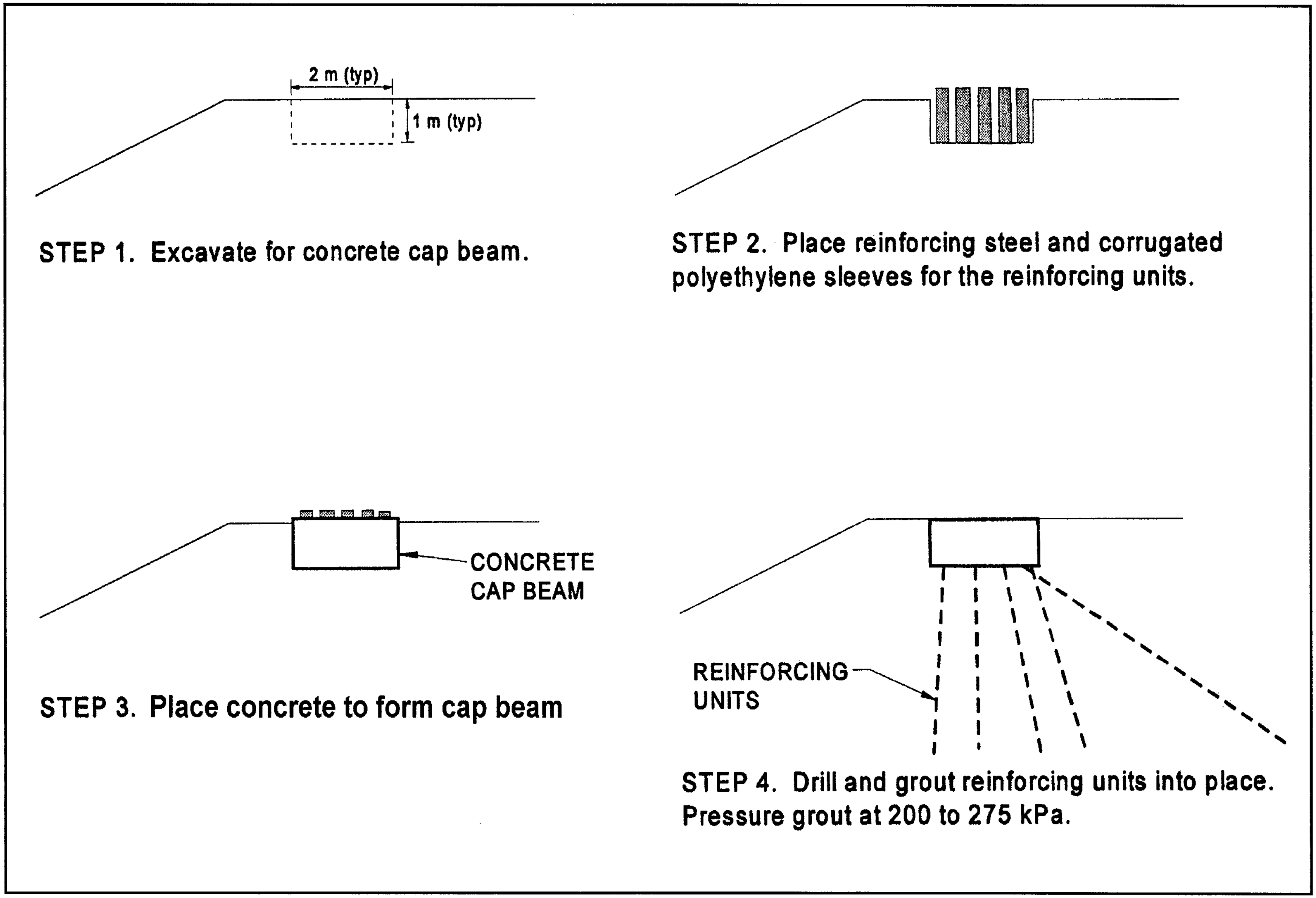


Figure 28. Construction sequence for Insert Walls^{SM} (after Sabatini et al. 1997)

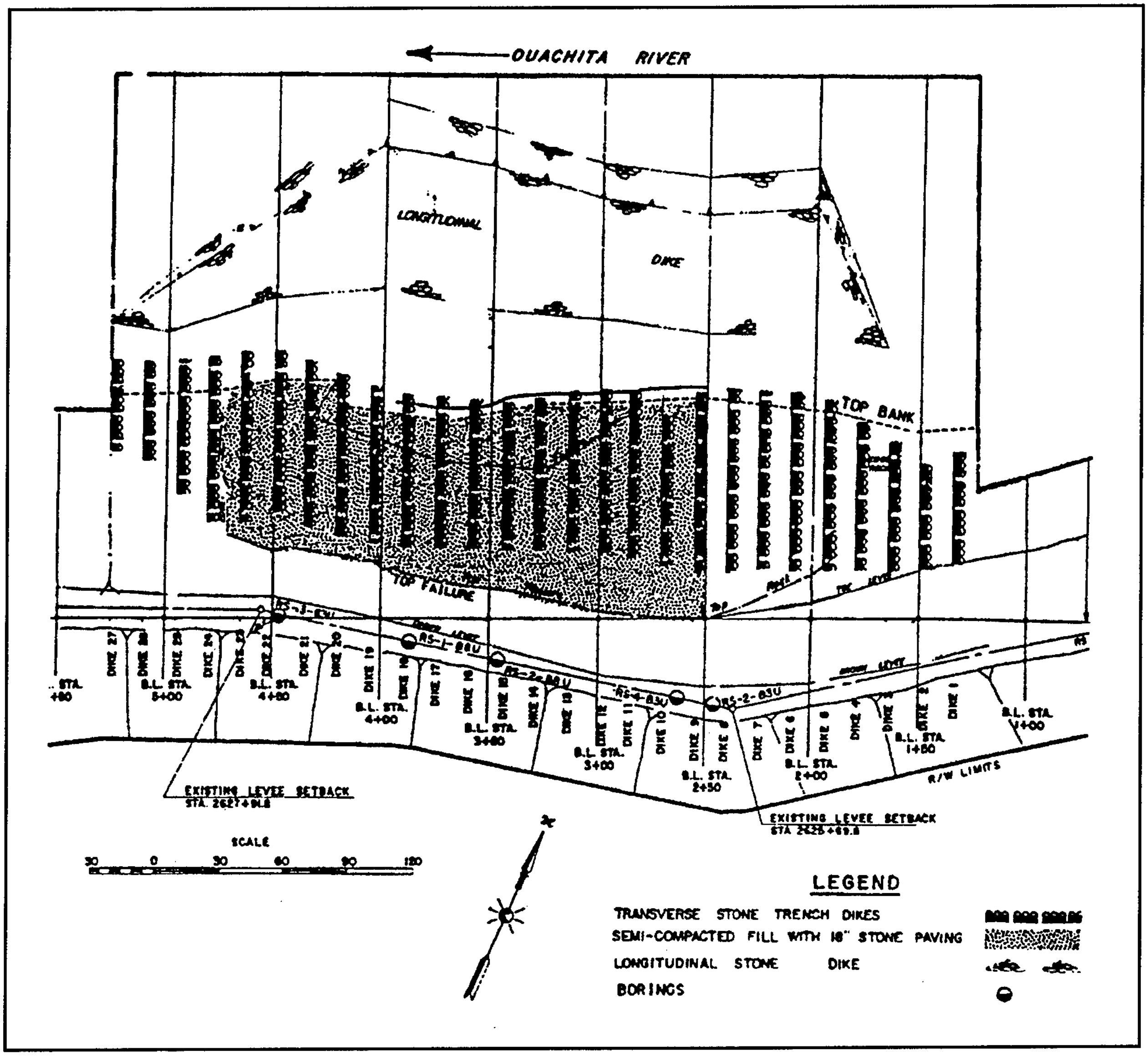


Figure 29. Site plan of stone-fill trenches used to stabilize levee on the Ouachita River at Rilla, LA, by USAED, Vicksburg, in 1988 (from Sills and Fleming 1994)

Advantages and disadvantages of stone-fill trenches are given in Table 5. The advantages of stone-fill trenches are a relatively low cost, rapid construction using readily available construction equipment (backhoe, bulldozer, and dump truck), and drainage of slope (reduces piezometric pressure on the failure surface and increase the stability) when an outlet is installed on the downstream end. Disadvantages are that stone must be available for use as backfill and arching of soil between trenches is not considered in design (conservative).

The design of stone-fill trenches involves a back analyses to determine an average shear strength acting on the failure surface producing a safety factor of unity. Although there is an infinite number of shear-strength parameters

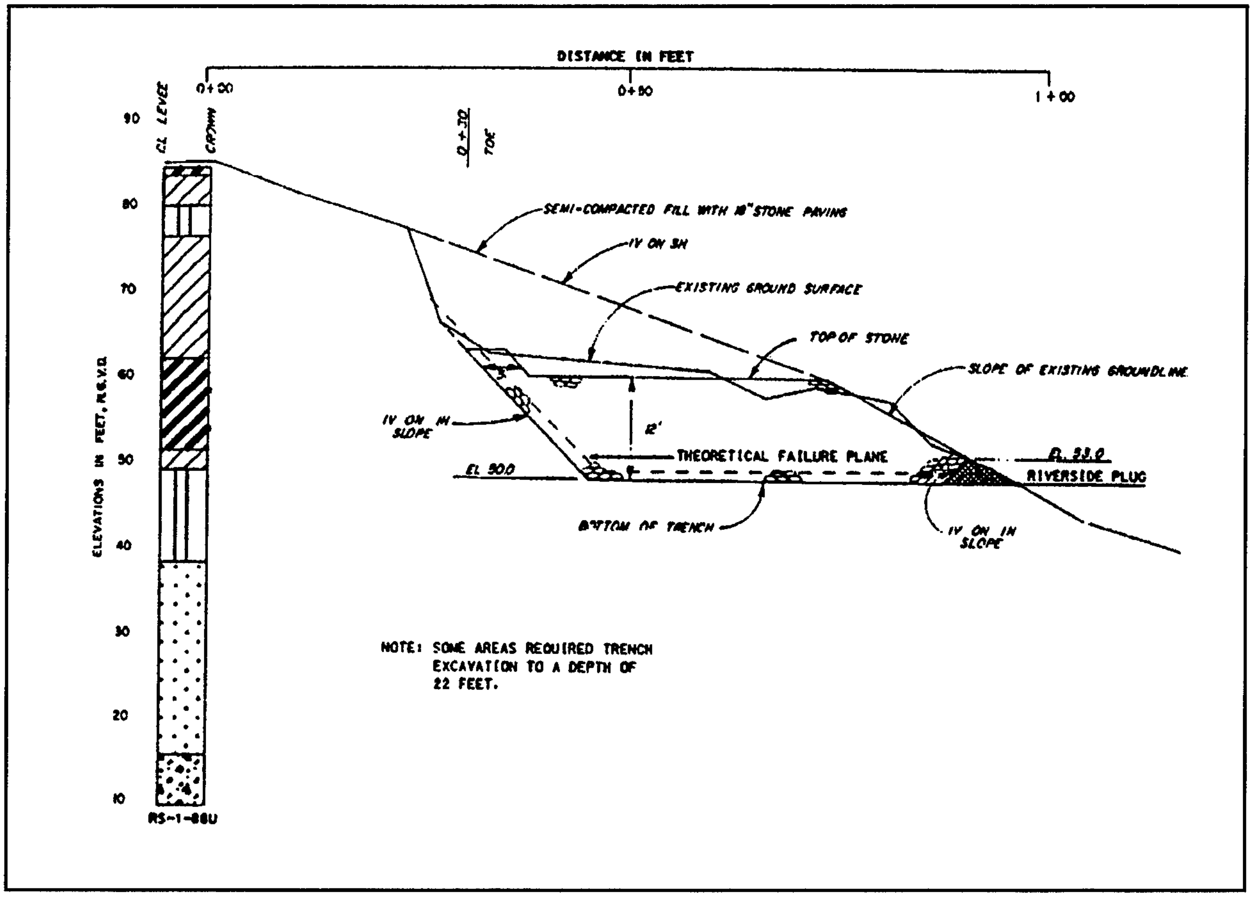


Figure 30. Section of slide area of stone-fill trenches used to stabilize levee on the Ouachita River at Rilla, LA, by USAED, Vicksburg, in 1988 (from Sills and Fleming 1994)

(cohesion and angle of internal friction) which will produce a safety factor of unity for a given slope, only one set of values will match the geometry of the failure surface (Wright, Isenhower, and Kayyal 1989). For example, Figure 31 gives values of total stress, shear-strength parameters corresponding to a factor of safety of unity for a circular-arc, critical failure surface passing through the toe of the slope (Abrams and Wright 1972). Knowing the width of the trench based on the construction equipment to be used (typically 2.5 ft for a backhoe), a trench spacing is computed to provide a composite shear strength required to raise the factor of safety to a desired value (usually 1.25). Then the center-to-center trench spacing is determined using a procedure detailed by Sills and Fleming (1994). As previously stated, due to arching of the soil between the trenches (not considered in the design analyses), the actual safety factor should be greater than the computed safety factor (Sills and Fleming 1992).

Construction of stone-fill trenches on the riverside of the levee generally begins downstream and proceeds upstream. A soil plug is left in place riverside of the trench. Existing topsoil is removed and stockpiled, and trench excavation is performed by a backhoe. Only a short length of unfilled trench is open at any time. The rock is placed (not compacted) to a 3-ft depth below the failure

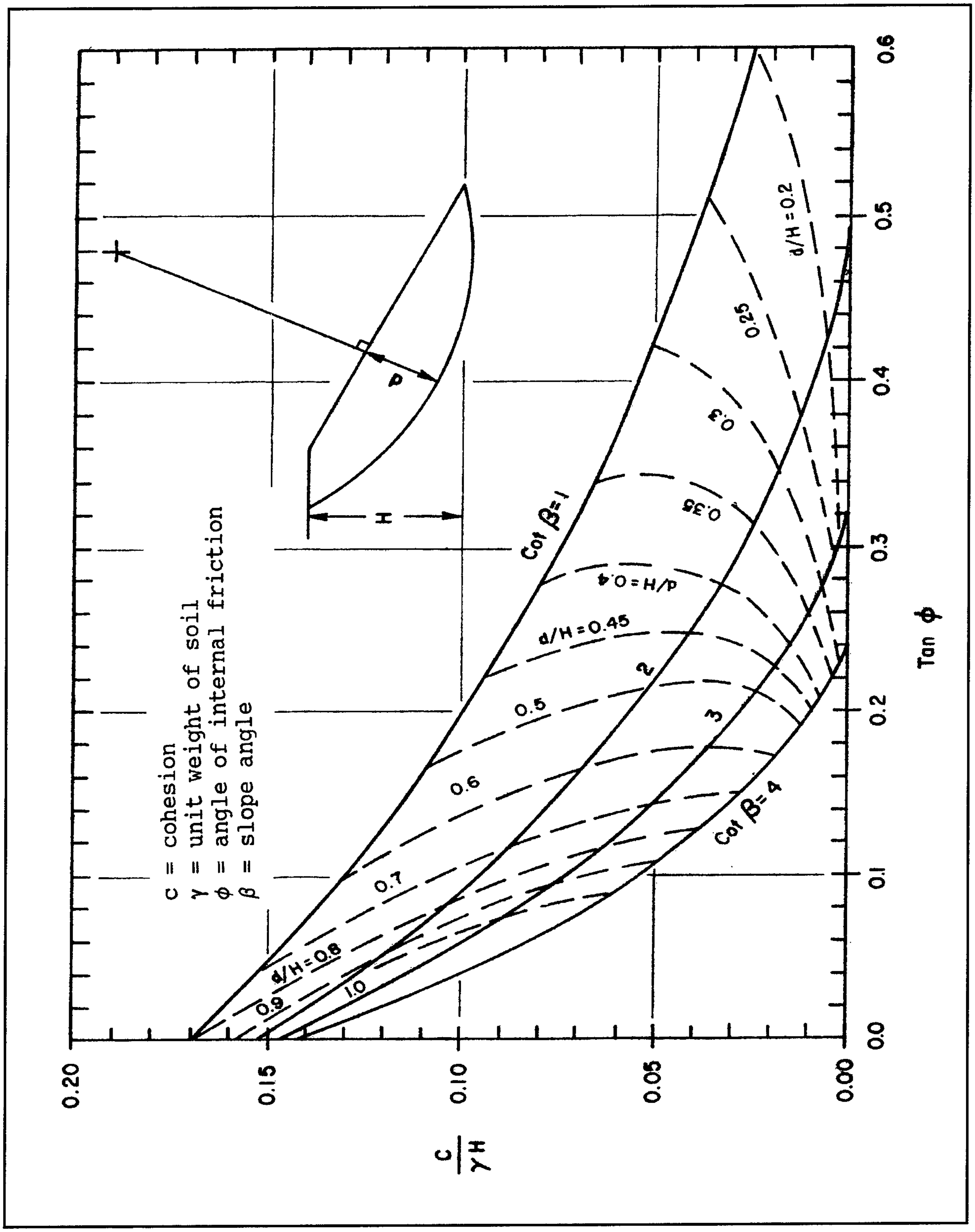


Figure 31. Values of total stress, shear-strength parameters corresponding to a factor of safety of unity for a circular-arc, critical failure surface passing through the toe of the slope (after Abrams and Wright 1972)

surface and filled to within 3 ft of the ground surface. Soil is used to fill the remaining portion of the trench, and erosion protection is provided as needed. Consideration should be given to placing an outlet at the proper elevation in the trench to reduce the piezometric pressure on the failure surface and increase the stability (Sills and Fleming 1992, 1994; Wardlaw et al. 1984).

In summary, stone-fill trenches are applicable for soils which will remain stable where trenches are excavated with vertical side slopes to a depth below the failure surface. It offers low cost and rapid construction, uses conventional, readily available light construction equipment, and provides drainage of the slope when an outlet is provided. Stone must be available for backfill, and arching of soil between trenches is not considered in design (conservative).

Randomly Distributed Synthetic Fibers

As stated previously, shallow slides typically occur 2 to 35 years (with an average of 18 years) following construction of levees with highly plastic clay. Weathering (repeated cycles of desiccation and wetting) produces shrinkage cracks. If rainfall runoff fills the cracks more quickly than they can swell closed, hydrostatic force due to the water exerts a lateral thrust. This lateral thrust, together with the decreased shear strength caused by softening of exposed surfaces in the cracks, may lead to shallow slope failure (Bromhead 1992).

One way to prevent slides from occurring is to protect the surface soil from the weathering process and thus reduce the tendency of the soil to crack. This can be done by providing a sand-gravel cover, modifying the soil with lime, etc. Also, fibers have been used to change the properties of soil (Shewbridge and Sitar 1985, Grogan and Johnson 1994). Discrete fibrillated polypropylene fibers have been used in slope stabilization for the past 8 years (Austin, Shrader, and Chill 1993).

Under the study herein, laboratory tests were conducted to assess the feasibility of using randomly distributed short polypropylene fibers to reduce the development of desiccation cracks in clay (Shulley, Leshchinsky, and Ling 1997). Although the fibers increased the tensile strength and were effective in reducing the amount of desiccation cracking, when subjected to wet/dry cycles attempting to simulate environmental conditions over time, the effectiveness of the fibers was not as evident. It was recommended that the structure of the fibrillated fibers be optimized through further research. Longer fibers with a different texture may provide increased cracking resistance under wet/dry cycles (Mercer et al. 1985).

In summary, randomly distributed synthetic fibers in their present configuration, i.e., short and smooth, are not recommended for slope stabilization in clay subjected to desiccation cracking.

Restraint Structures

Piling and drilled shafts placed in row(s) to provide lateral restraint (Figure 32) are a viable option to stabilize small surface slides in cohesive soils where movement (typically failure) has occurred and where there is a competent underlying strata (lower part of the levee or foundation) within about 20 ft or less (Hopkins et al. 1988; Oakland and Chameau 1986). Precast concrete wall panels may be affixed to the top of the drilled shafts, referred to as a slide suppressor wall, thus providing further lateral resistance and precluding soil movement around and between the drilled shafts as shown in Figure 33 (Abrams and Wright 1972; Isenhower, Wright, and Kayyal 1989). If necessary, a drainage system should be installed behind the wall (Nethero 1982).

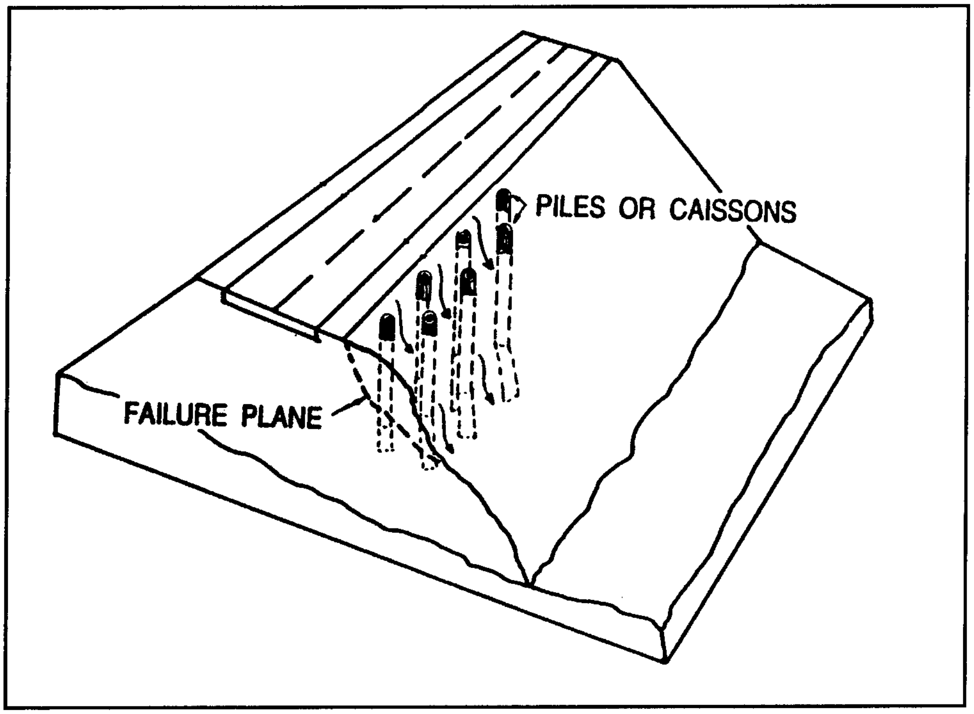


Figure 32. Restraint structure used to stabilize slope (from Hopkins et al. 1988)

For driven piles, resistance is mobilized soon after driving (allowing for dissipation of excess pore water pressures), while drilled shafts require time to acquire full strength. Pile driving may preclude the use of restraint structures on actively moving landslides (Bromhead 1992). However, drilled shafts can be installed without decreasing slope stability during construction (Nethero 1982).

Advantages and disadvantages of restraint structures are given in Table 5. The advantages of restraint structures are rapid construction and, in cases where slope failure has not occurred, drilled shafts do not require excavation which would create additional load on a distressed slope (Morgenstern 1982). A disadvantage is that construction cost increases rapidly with increased height of

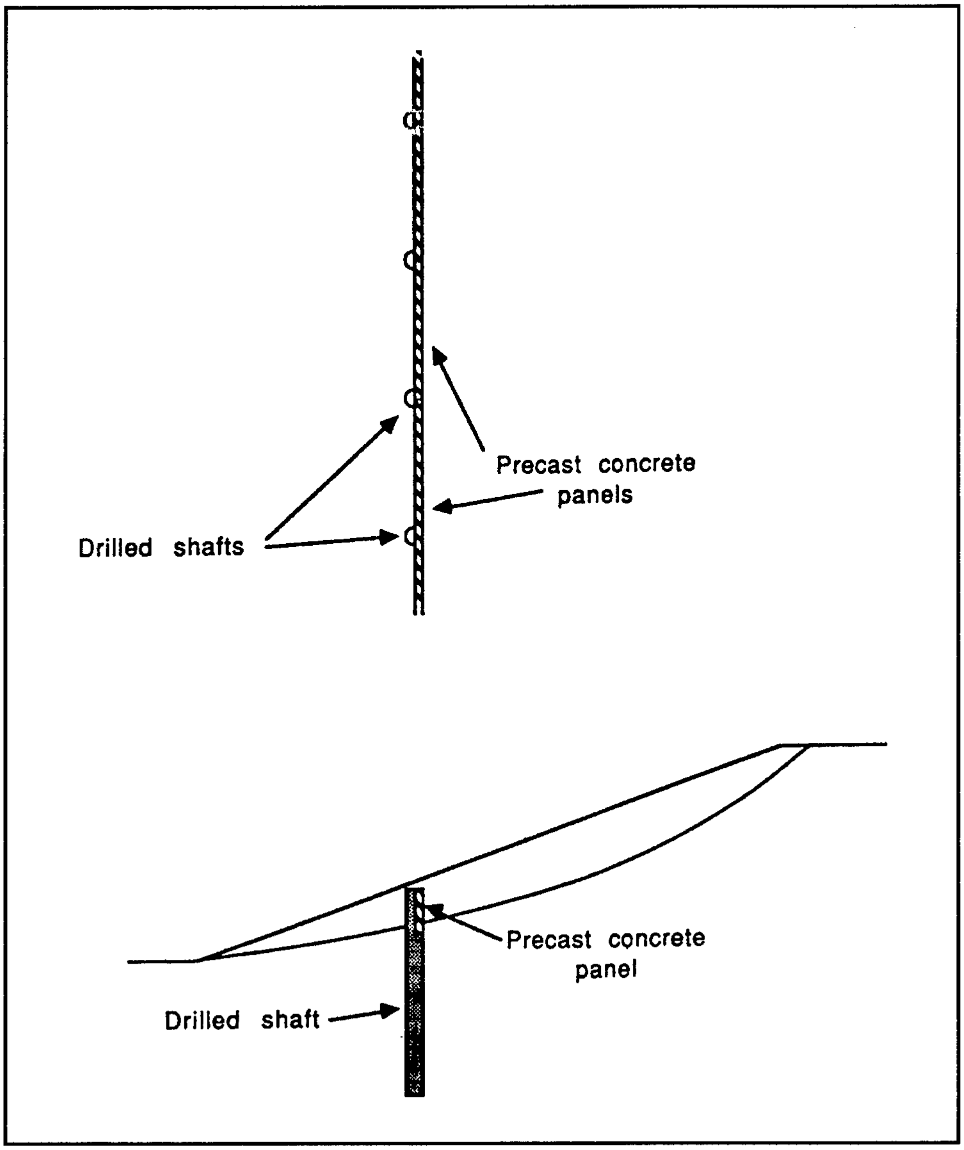


Figure 33. Slide suppressor wall with precast concrete wall panels to retain the soil (courtesy of Transportation Research Board, Isenhower, Wright, and Kayyal 1989)

the wall (Nethero 1982; Abramson et al. 1993). Restraint structures may be made more effective and less costly by anchoring into a competent strata or using anchor blocks (Nethero 1982; Hopkins et al. 1988).

Critical factors to consider with restraint structures are (Hopkins et al. 1988):

a. For restraint structures without panels (Figure 32), pile spacing between individual piles should be less than 3 ft to prevent flow of soil around the piles (Nethero 1982). Cohesive soils with a plasticity index greater than 30 and liquid limit greater than 50 do not mobilize the arching effect sufficiently to prevent flow of soil around the piles, and restraint

structures without panels should not be constructed in these soils (Abramson et al. 1993).

b. Piles or drilled shafts should be anchored into a competent underlying strata (one-third of the pile length for soil and one-quarter for rock) and extend across the width of the failure surface and several sections past the flank of the slide into stable material (Nethero 1982).

c. Typically, the restraint structure is located near the toe of the slide at one-third of the slide height (Chugh 1982; Isenhower, Wright, and Kayyal 1989; Wright, Isenhower, and Kayyal 1989).

d. Driving forces exerted on the restraint structure increase as a function of the height squared. The failure mass should not be much larger than a wedge of soil with a failure plane rising at a slope of about 1H:1V.

A design procedure for slide suppressor walls with 18- or 24-in.-diam drilled shaft and precast concrete wall panels (see Figure 33) is as follows (Isenhower, Wright, and Kayyal 1989; Wright, Isenhower, and Kayyal 1989):

a. A drilled shaft length and wall panel width are selected from Figures 34 or 35. The wall panel height is set equal to the depth of the slide (vertical distance from the original ground surface to the failure surface) plus 1 ft. The top of the wall panel is set slightly below the ground surface, and the base of the wall panel is about 1 ft below the failure surface.

b. An earth pressure coefficient of unity, indicating hydrostatic stresses, can be assumed for slopes 2H:1V or flatter in cohesive soils that have failed or are barely stable and where the slide depth and wall height do not exceed one-third of the height of the slide (Wright, Isenhower, and Kayyal 1989).

c. The precast concrete wall panel is designed based on the panel length, height, and thickness (Isenhower, Wright, and Kayyal 1989).

d. Various drilled shaft lengths and wall panel widths can be tried and the lowest unit cost selected. The cost of the slide suppressor wall is largely determined by the cost of the drilled shafts with the cost of the precast concrete wall panels having a secondary influence. Higher walls require increased depth of penetration into competent underlying strata, greater drilled shaft diameter, and additional reinforcing steel. Therefore, the construction cost increases rapidly with increased height of the wall (Abramson et al. 1993). Slide suppressor walls may be made more effective and less costly by anchoring into a competent strata or using anchor blocks (deadman anchors) (Hopkins et al. 1988).

A design procedure for a restraint structure without wall panels (Figure 32) is given by Chugh (1982). Additional information relative to the design of drilled piers for slope stabilization including a finite element analysis with

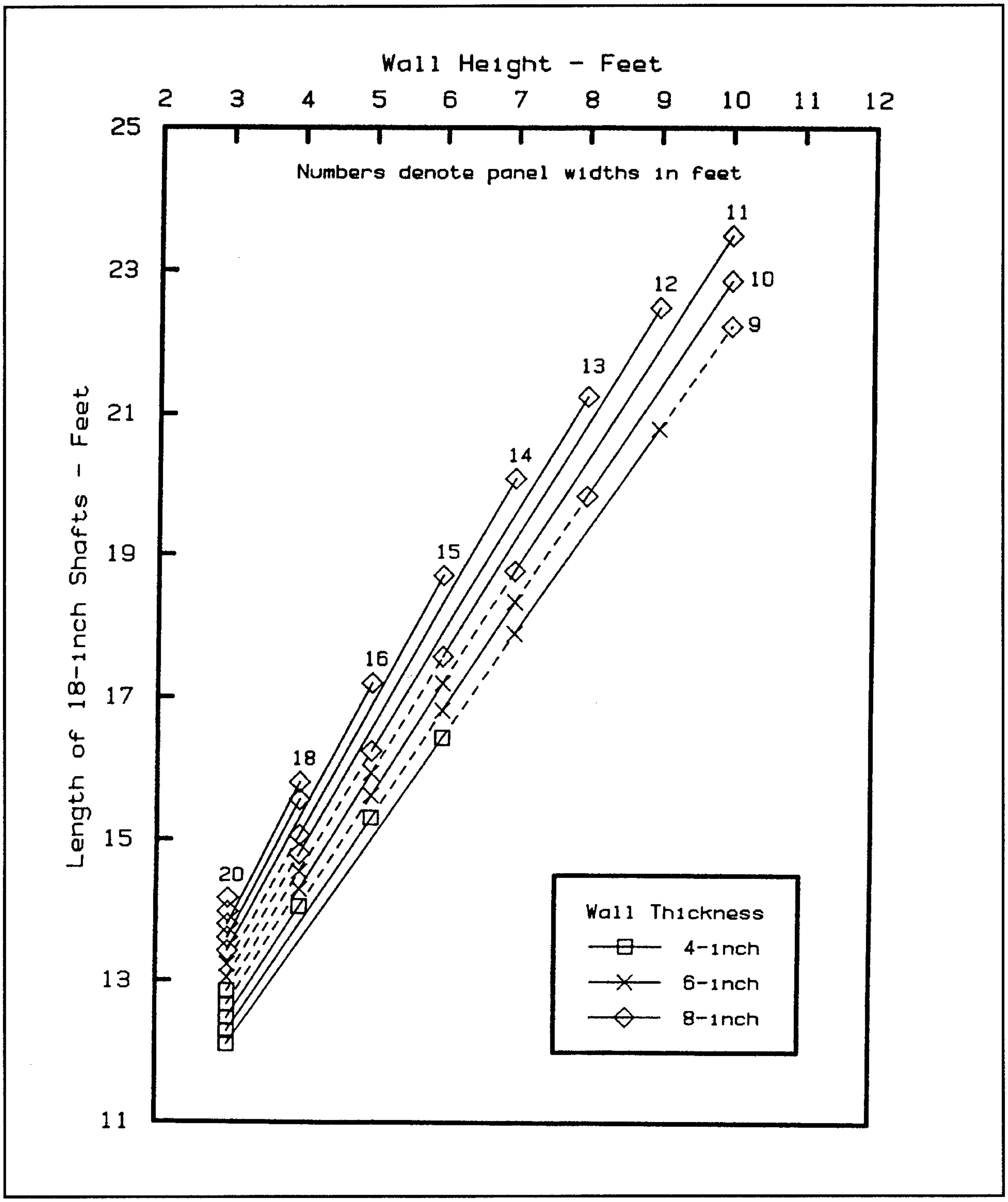


Figure 34. Design chart for slide suppressor wall with 18-in.-diam drilled shafts and precast concrete wall panels (courtesy of Transportation Research Board, Isenhower, Wright, and Kayyal 1989)

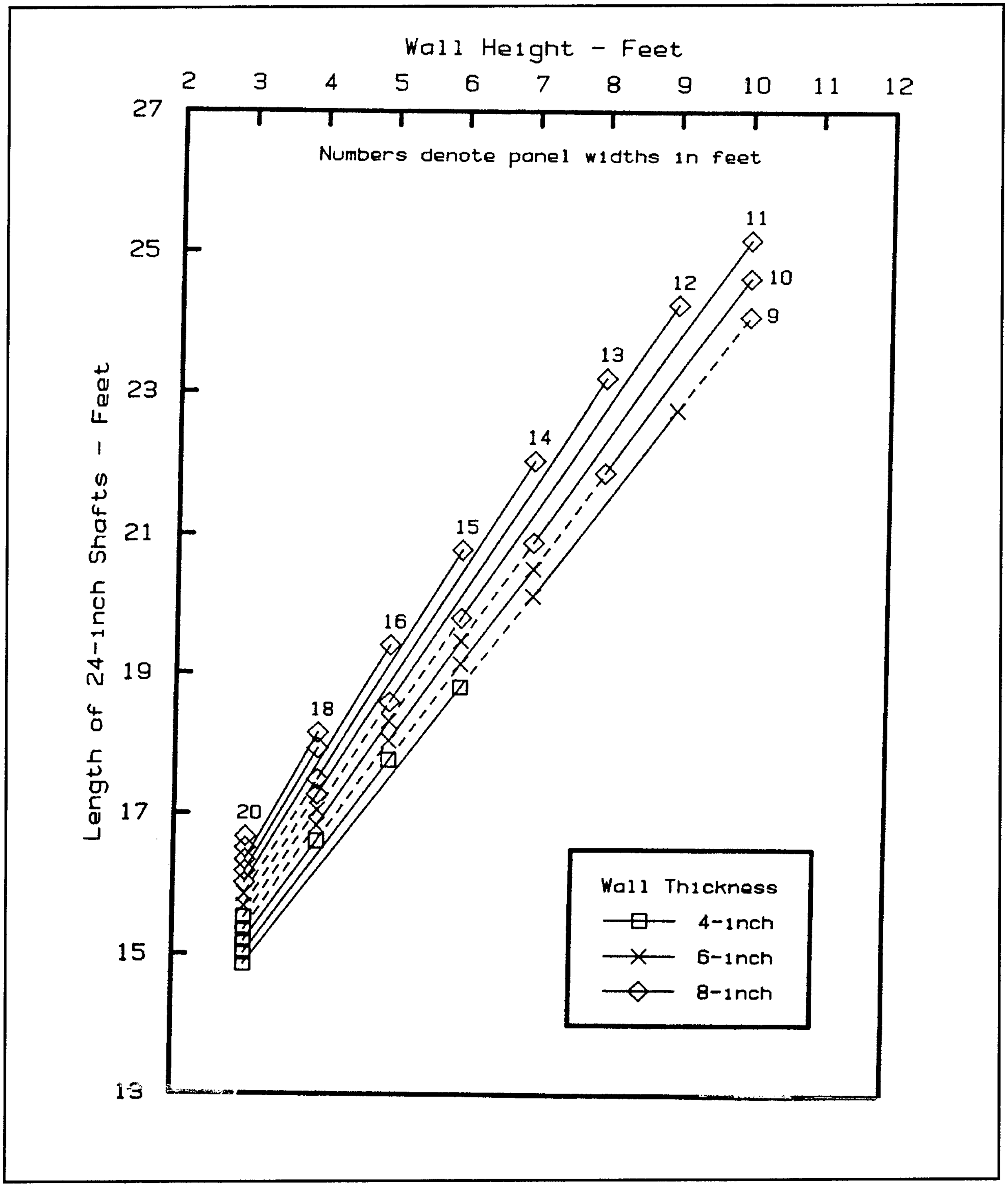


Figure 35. Design chart for slide suppressor wall with 24-in.-diam drilled shafts and precast concrete wall panels (courtesy of Transportation Research Board, Isenhower, Wright, Kayyal 1989)

nonlinear soil behavior, construction sequences, and consideration of arching between piers is available (Oakland and Chameau 1986).

Alternative forms of construction may be used. For example, the restraint structure may use steel I-beams or H-piles driven into place or placed into predrilled holes and backfilled with concrete or available material such as railroad rails in predrilled holes and backfilled with concrete, sand, or gravel (Abrams and Wright 1972; Hopkins et al. 1988).

Geosynthetic Drainage System

As stated previously, in levees constructed of plastic clay, weathering produces shrinkage cracks down to the desiccation depth (5 to 7 ft). If rainfall runoff fills the cracks more quickly than the cracks can swell closed, hydrostatic force due to the water exerts a lateral thrust. This lateral thrust, together with the decreased shear strength caused by softening of exposed surfaces in the cracks, may lead to shallow slope failure (Bromhead 1992). One way to prevent this from occurring is through a geosynthetic drainage system.

Geocomposite drains are prefabricated drainage systems made wholly or partially of polymeric materials. One type of geocomposite drain is the edge drain (Figure 36). The edge drain has been used since the 1970's as land drains, highway edge drains, structural drains behind retaining walls, and horizontal drains in embankments (Fritsch and Prodinger 1980, Murray and McGown 1992, Hunt 1993, Broms 1993).

Under the levee rehabilitation study, an innovative method was devised to use a geocomposite drainage system to remove runoff water from the cracked desiccated upper zone of the levee as a temporary measure to improve the surficial stability of levee slopes. The proposed use of a geosynthetic drainage system to remove runoff water entering a levee through a network of interconnected desiccation cracks is an extrapolation of existing practices. Therefore, this method is considered a temporary measure until verified by full-scale field testing (Leshchinsky 1996). Plastic clays, for which the geosynthetic drainage system is proposed, will remain stable for trenches excavated with vertical side slopes to a depth below the failure surface. However, the Occupational Safety and Health Administration (OSHA), U.S. Department of Labor, USACE, safety requirements must be met if workers are required in the open trenches.

Drainage is often the most cost-effective method for slope stabilization (Kleppe and Denby 1984; Olcese, Vescovo, and Fantini 1990; Abramson et al. 1996). The edge drain with its relatively low cost and ease of installation has been used extensively since 1985 as a drain at the edge of pavements to convey water from the <u>intact soil</u> underneath the road (Cherubini et al. 1992; Elsharief 1992; Perez 1993). Although the physical composition of the drain is the same, the use of a geosynthetic drainage system to improve the surficial stability of levee slopes involves removing runoff water from a <u>system of cracks and fissures</u> intersecting the drain.

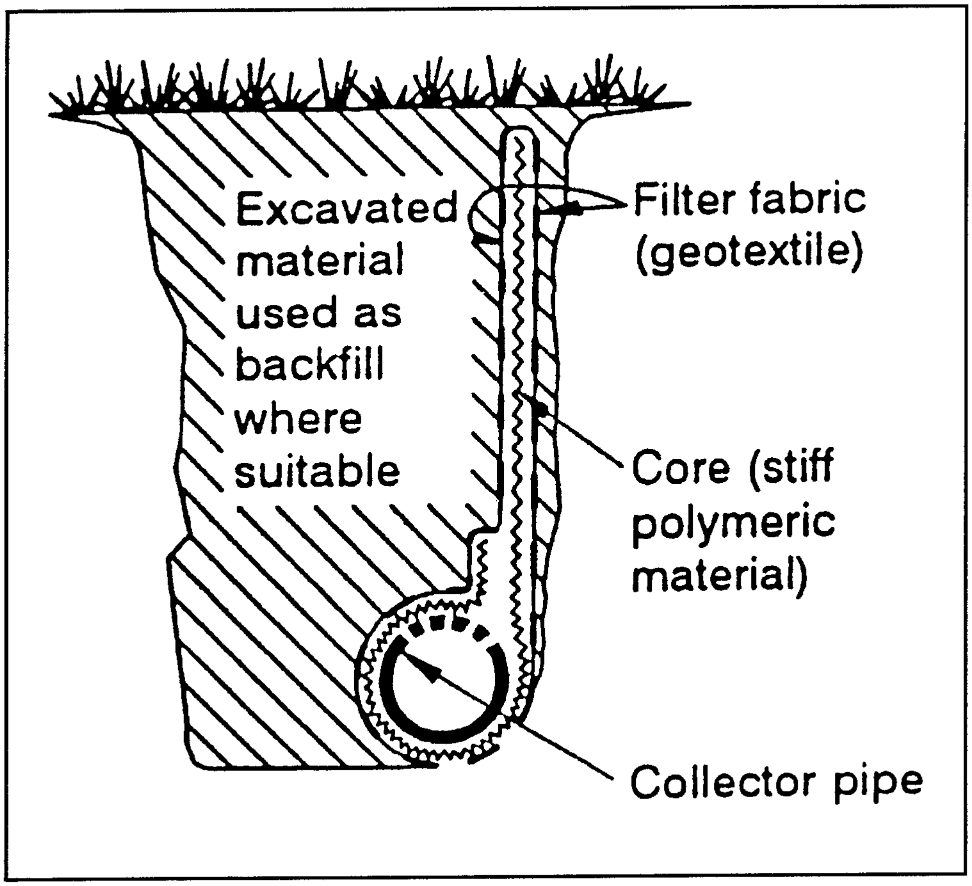


Figure 36. Edge drain (from Leshchinsky 1996)

Advantages and disadvantages[1] of geosynthetic drainage systems are given in Table 5. The advantages of geosynthetic drainage systems are the relatively low cost and automated method of installation. Disadvantages include possible smear during excavation of the upper side of the trench at the interface between the cracked soil and drain, possible piping of soil particles through the geotextile and into the drain, and possible blockage of outlets by silt accumulation and/or grass root intrusion (Kleppe and Denby 1984; Murray and McGown 1992; Kearns 1992, 1995; Elsharief 1992; Koerner, Koerner, and Fahim 1993; Perez 1993).

The design of a geocomposite drainage system to remove water from the cracked upper zone of a levee is given by Leshchinsky (1996). Figure 37 shows a schematic view of a geocomposite drain installed as an interceptor to remove water from the cracked upper zone of a levee. The drain is placed against the upper side of the excavated trench to assure direct contact interface with the cracks and facilitate drainage into the trench (this is in contrast to the use of the

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[1] Since the method of using geosynthetic drainage system to improve the surficial stability of levee slopes has not been tried, the advantages and disadvantages are hypothetical.

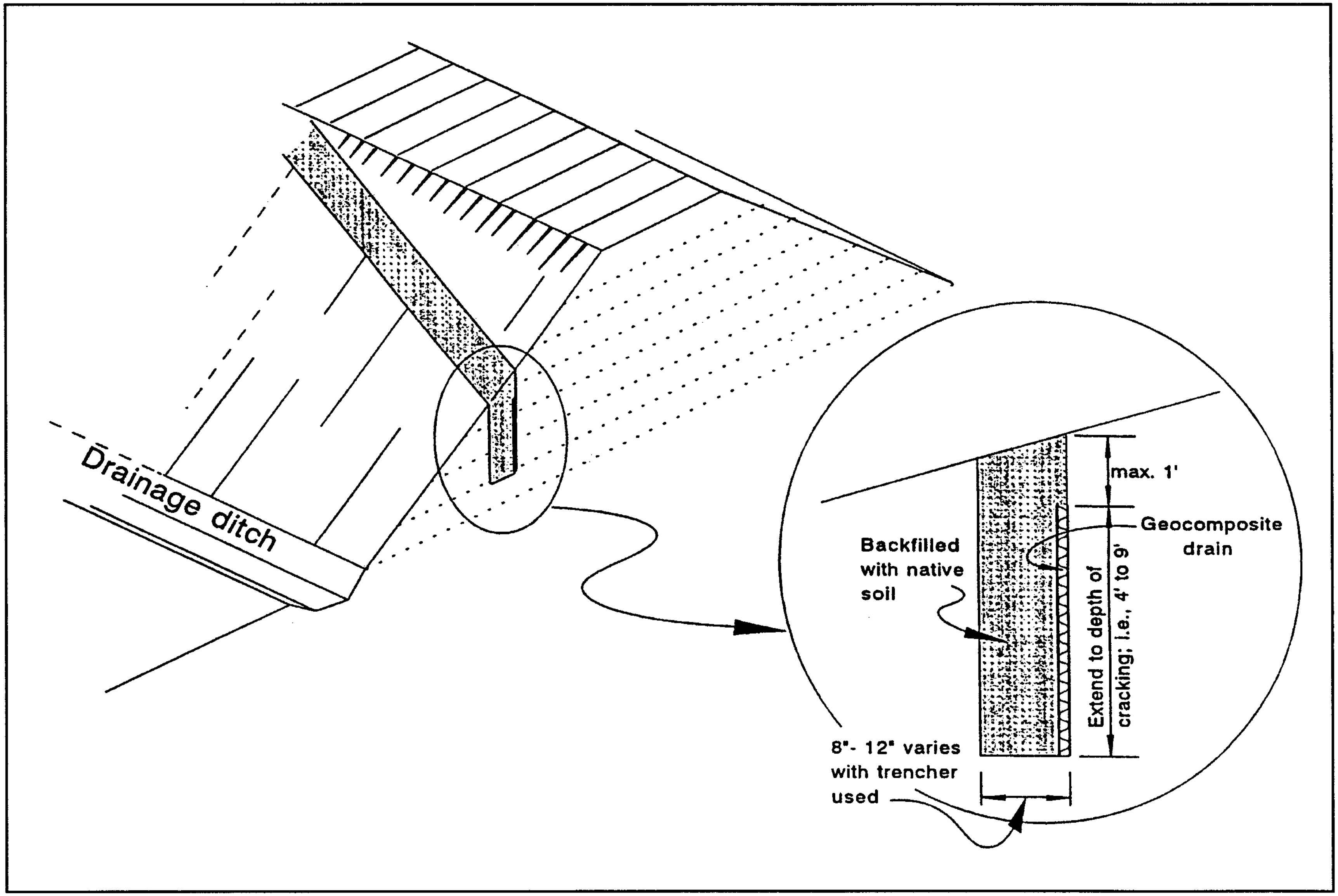


Figure 37. Schematic view of a geocomposite drain installed as an interceptor to remove water from the cracked upper zone of a levee (from Leshchinsky 1996)

geocomposite drain as a highway edge drain where it is recommended that the geocomposite drain be placed on the side of the excavated trench away from the direction of flow to avoid spaces between the base soil and geocomposite drain (Koerner, Koerner, and Fahim 1993). Using a plan view of the levee, the desired layout of the geocomposite drainage system is selected as shown in Figures 38 to 40. Detailed design instructions are given by Leshchinsky (1996).

The construction sequence for a geocomposite drainage system to remove water from the cracked upper zone of a levee is given in Figure 41. An unsupported trench is first excavated to a depth below desiccation cracking. The drain, either a sheet drain or corrugated tubing drain, is then placed against the upper side of the trench and temporarily supported by stakes. The excavated soil is then placed and backfilled to just under the top of the drain. Topsoil is backfilled to the surface and the embankment is seeded over the trench. As previously stated, the outlet for the geocomposite drain (Figure 42) must be designed, constructed, and maintained to avoid possible blockage by silt accumulation and/or grass root intrusion (Leshchinsky 1996, Murray and McGown 1992).

In summary, a geocomposite drainage system could be used as a temporary measure to remove runoff water from the cracked desiccated upper zone of the levee and thus improve the stability of the slope. It offers low cost and use of automated construction equipment. Possible disadvantages include smear of the upper side wall during excavation of the trench, piping of soil particles through the geotextile and into the drain, and blockage of outlets by silt accumulation and/or grass root intrusion.

Lime-Fly Ash Injection

As stated previously, in levees constructed of plastic clay, weathering produces shrinkage cracks down to the desiccation depth (5 to 7 ft). If rainfall runoff fills the cracks more quickly than they can swell closed, the resulting lateral thrust and loss of shear strength may eventually lead to shallow slope failure (Bromhead 1992). Lime stabilization has been used successfully since 1974 to repair levee slides where failure was due to rainfall runoff into shrinkage cracks. Lime stabilization involves removing the failed material, treating it with lime (for more plastic clays a double application), and replacing at the original slope angle using limited compaction (Townsend 1979, Alvey 1994).

Since 1975, fly ash, a waste by-product of coal-burning power plants, combined with lime and water and injected into the levee slope, has been used as a slope remediation method (Pengelly and Holloway 1993; Joshi, Natt, and Wright 1981; Ferrell, Arman, and Baykal 1988; Blacklock and Wright 1986; Holloway 1994). When the slurry is injected into the levee (as described below) it migrates through and fills existing shrinkage cracks creating a 3-D network of lime-fly ash seams. In cases where a slide failure has occurred and the soil is removed and compacted to reconstruct the slope, hydraulic fracturing will

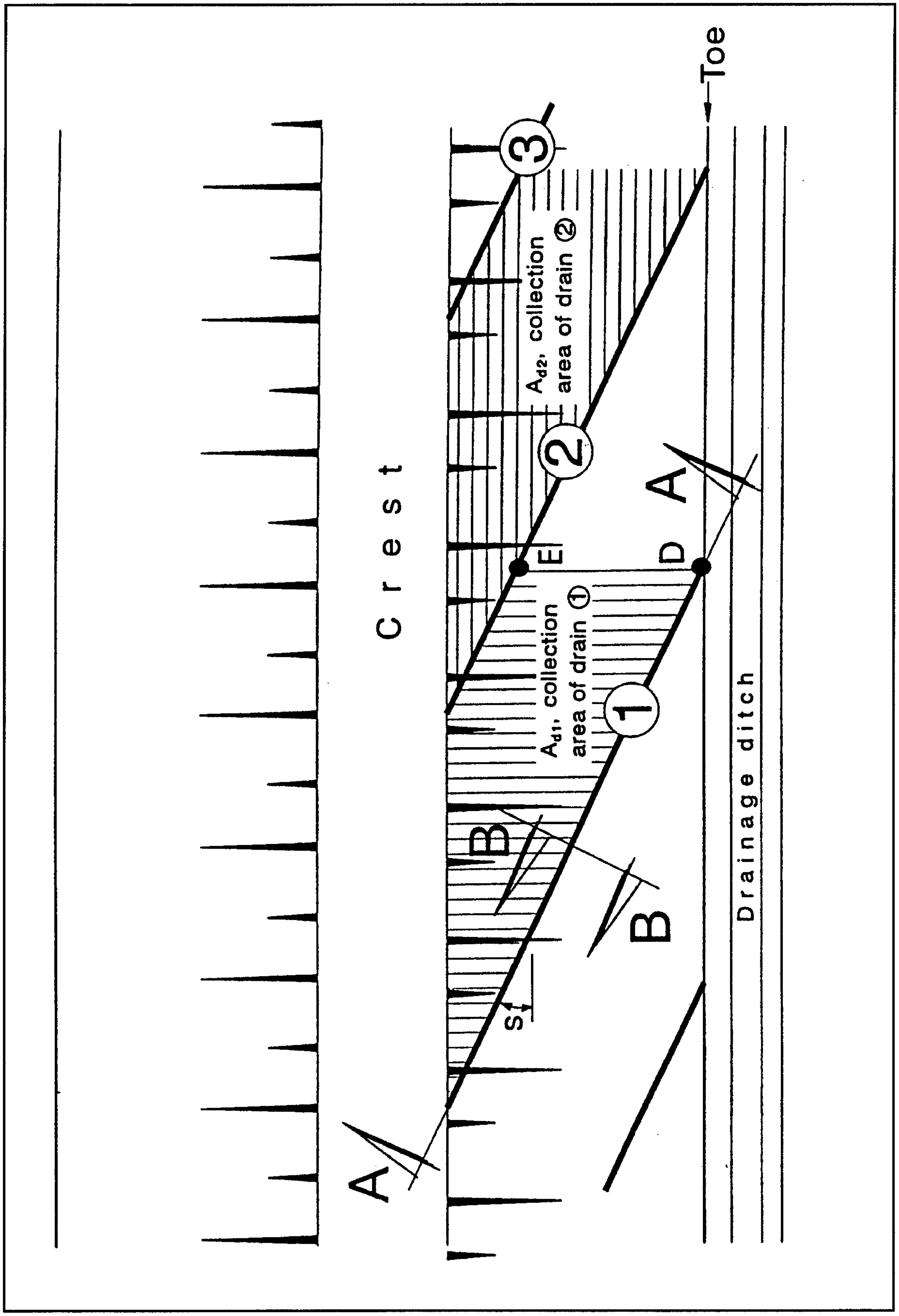


Figure 38. Plan view of levee with presumptive layout of geocomposite drainage system (from Leshchinsky 1996)

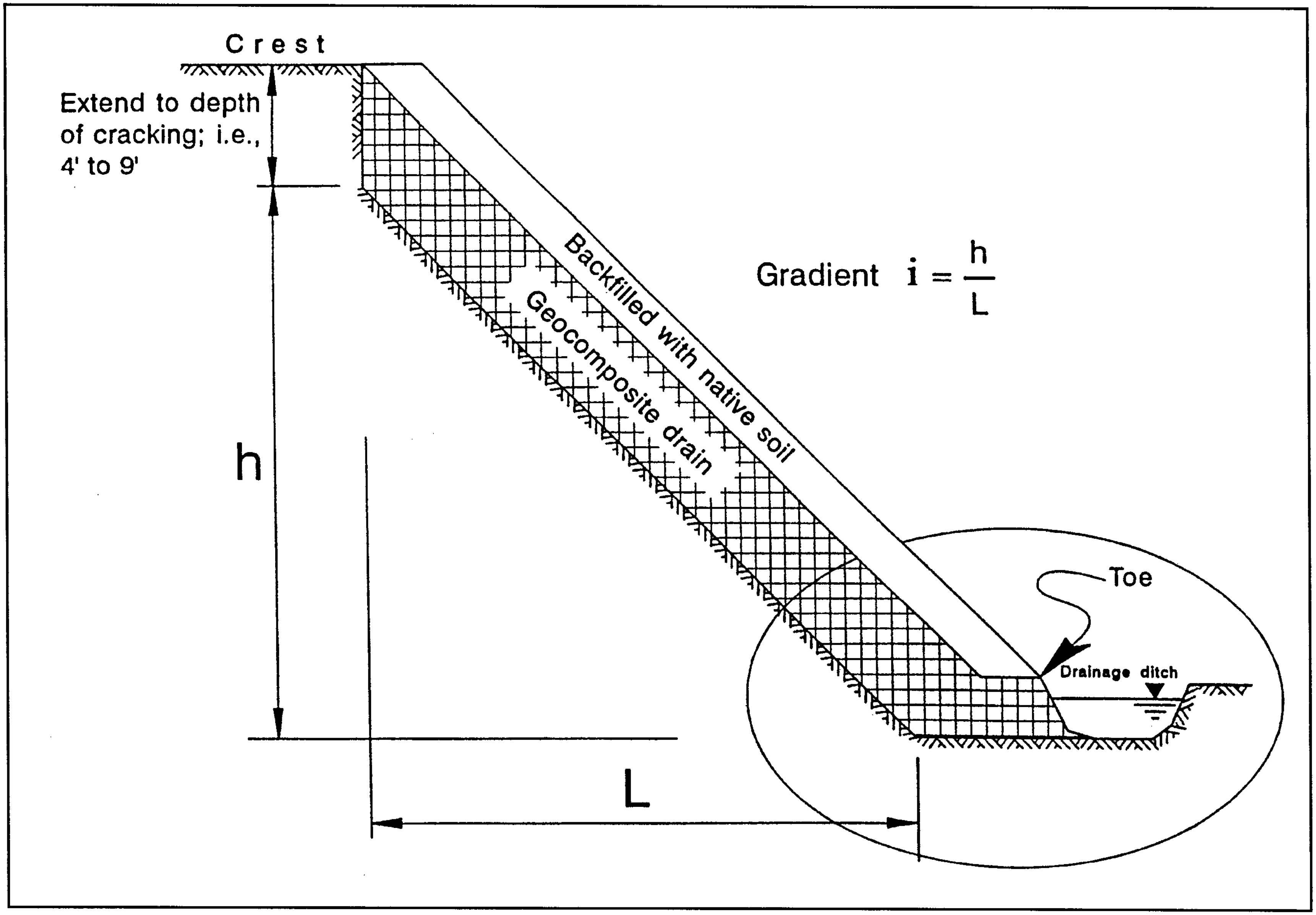


Figure 39. Section A-A (from Figure 38) along installed geocomposite drain (from Leshchinsky 1996)

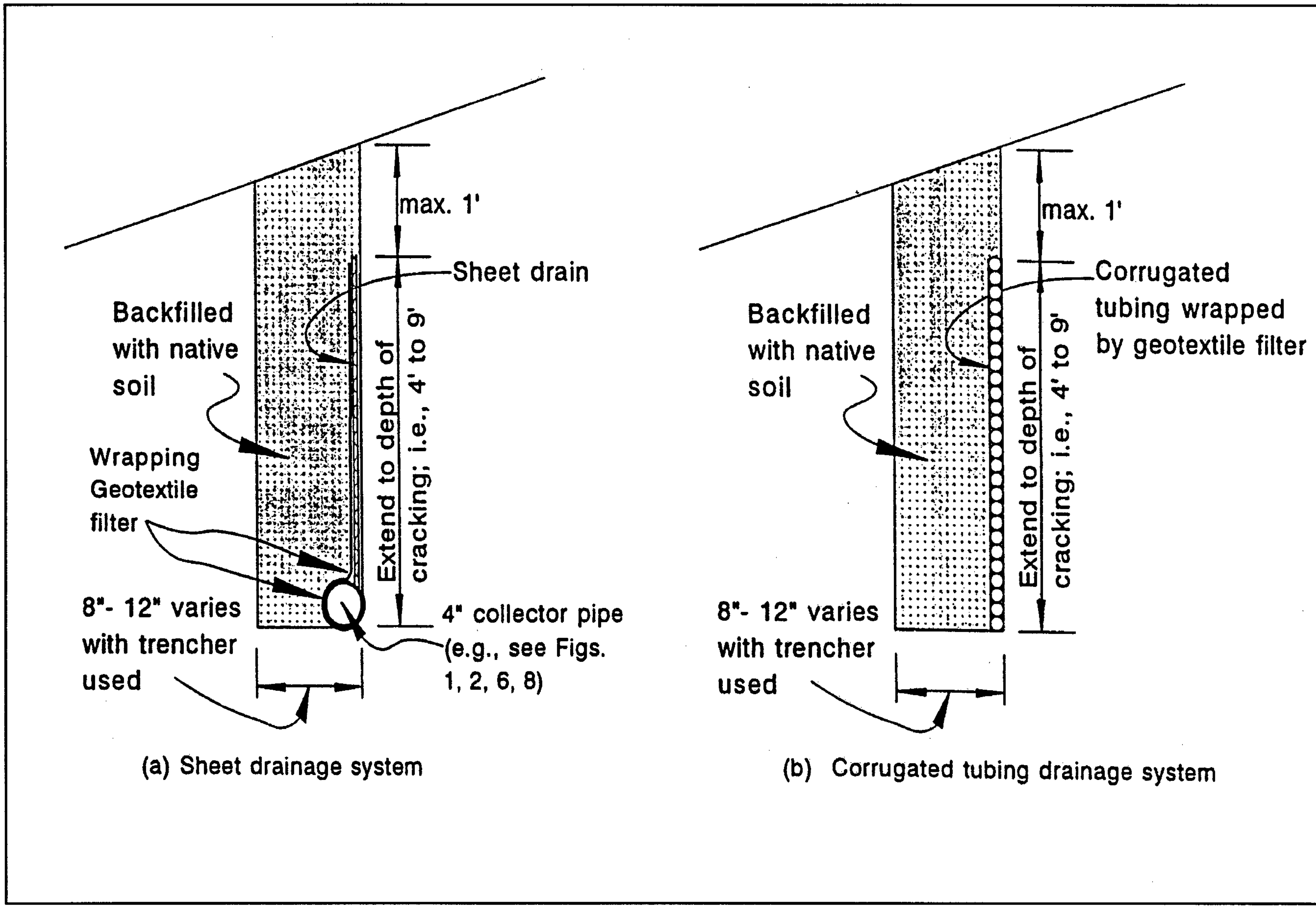


Figure 40. Section B-B (from Figure 38) perpendicular to drainage trench (from Leshchinsky 1996)

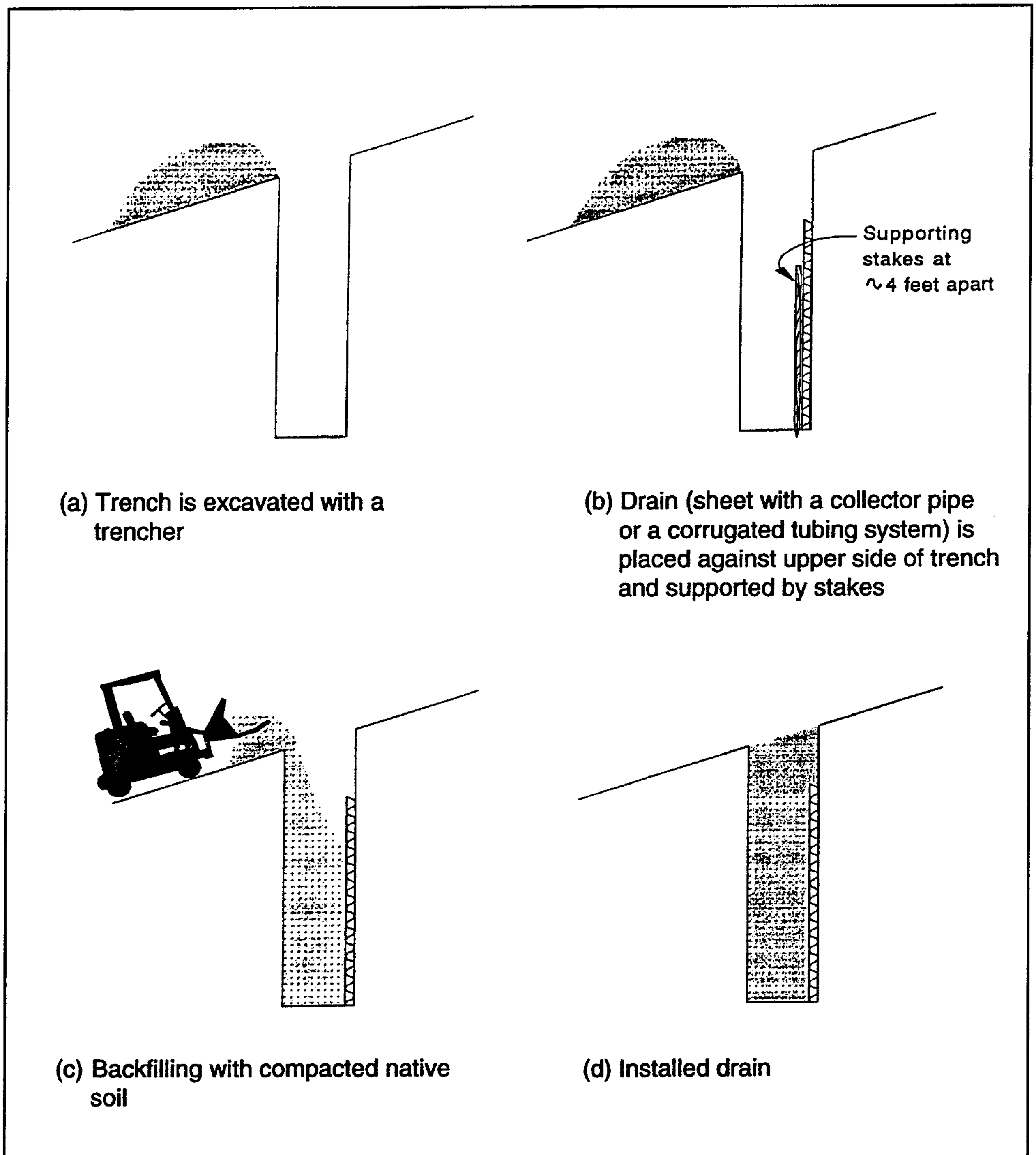


Figure 41. Construction sequence for a geocomposite drainage system (from Leshchinsky 1996)

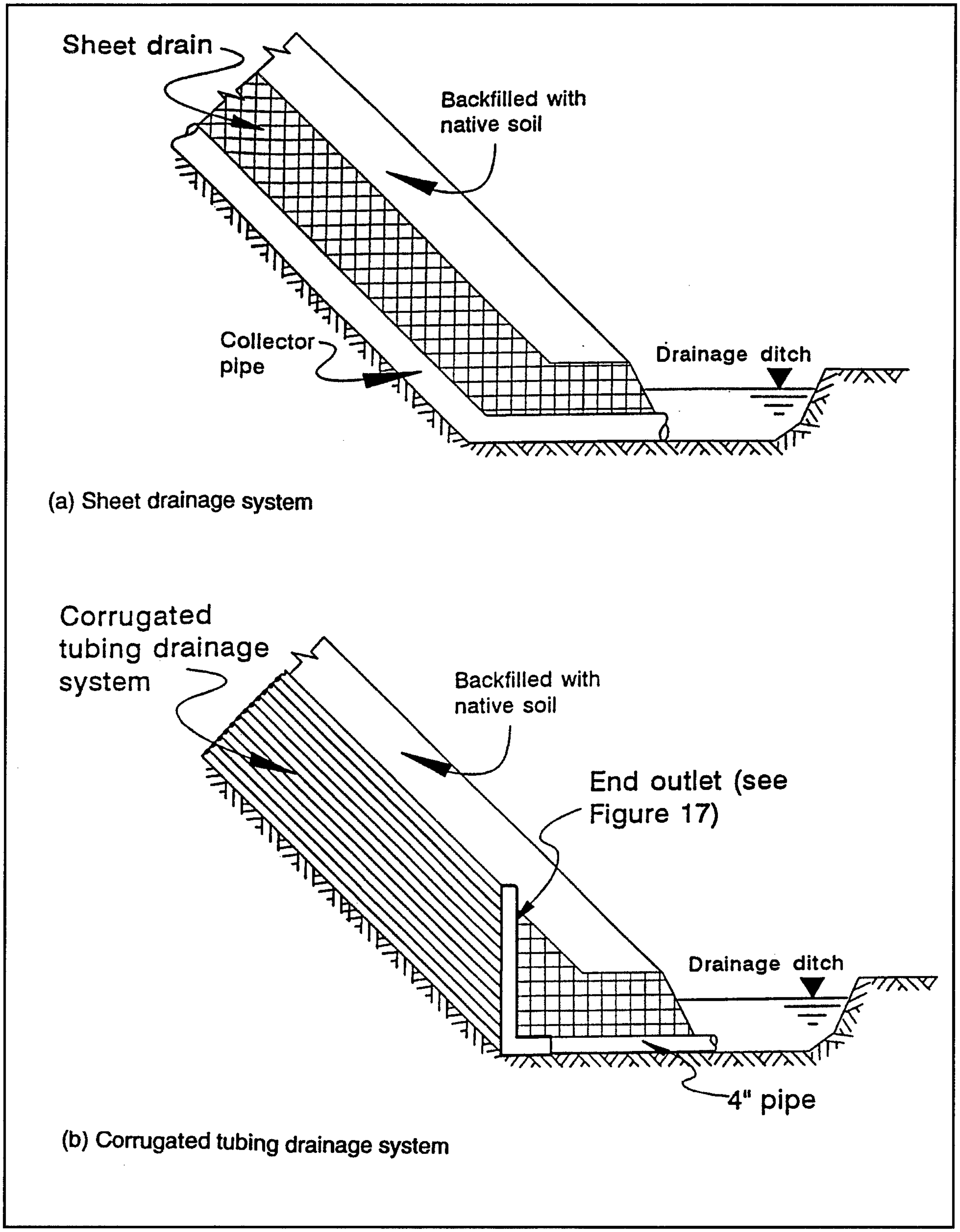


Figure 42. Outlet for geocomposite drainage system (from Leshchinsky 1996)

probably occur during the injection process, since the injection pressures (50 lb/sq in or greater) are larger than the at-rest lateral stresses in the soil, and the slurry will migrate through and fill the newly made cracks.

The network of lime-fly ash seams serves two functions: preventing surface runoff from entering and traversing the cracks and increasing (by 15 to 30 percent, depending on the method of comparison, as described below) the overall shear strength of the levee soil. Both of these functions serve to prevent slope failure over the short term. Preventing surface runoff from entering and traversing the cracks in the soil has a greater influence on slope stability than the increase in overall shear strength (Baez 1988). Available data from existing lime-fly ash injection of levees, first used in 1984, indicate that although shrinkage cracks occur in the lime-fly ash injected slopes, no slope failures have occurred (Perlea 1997).

Lime-fly ash injection is accomplished using an injection vehicle (truck or tractor) with several front-mounted injectors as shown in Figure 43. The injector rods, which are mounted 5 ft apart, are pushed into the soil in depth intervals of approximately 12 to 18 in. Typically, lime-fly ash is proportioned at the ratio of one part lime (quicklime slaked into hydrated lime) to three parts fly ash (Type C), although this should be varied as a function of soil properties. The lime and fly ash is then mixed into the slurry at 6 to 8 lb lime-fly ash per gallon of water. The injector rod tips disperse lime-fly ash slurry in a 360-deg pattern until refusal (soil will take no more slurry, and slurry is running freely on the surface around the injector rods, from previous injection holes, or from cracks at the surface). Injection pressures are typically within 50 to 70 lb/sq in., but there may be instances when the pressures approach 200 lb/sq in. The lime-fly ash slurry is continuously running throughout the injection process. The injection sequence begins at the toe of the landside slope and proceeds upward and then at the toe of the riverside slope and proceeds upward. In an attempt to eliminate the void left by withdrawal of the injector rod, injections at some locations have begun at the top of the levee and proceeded down slope. After 48 hr, secondary injections, between the primary injections, are performed. The depth of injection varies with soil conditions, typically ranging from 5 to 20 ft deep (Figure 44). As the lime-fly ash slurry is injected into the soil under pressure, the slurry follows the path of least resistance, moving through shrinkage cracks, tension cracks, compaction planes, root lines, sand lenses, or fractures created by the injection process. The slurry is deposited in horizontal seams often interconnected with vertical or angular veins (Wright 1973).

Normally the excess lime-fly ash slurry on the surface runs to the levee toe and is collected for disposal. In some cases, the levee slopes are disced to a depth of 6 in. to incorporate the excess lime-fly ash slurry remaining on the surface of the levee. Although discing the lime-fly ash slurry into the surface would increase the erosion resistance and strength of the soil, it would not prevent water from entering the levee surface because the lime would increase the permeability of the soil. Since lime typically raises the ph of the soil from 11 to 12, topsoil is added to the face of the slope to aid in the establishment of vegetation. All proof rolling and compaction is done perpendicular to the axis of

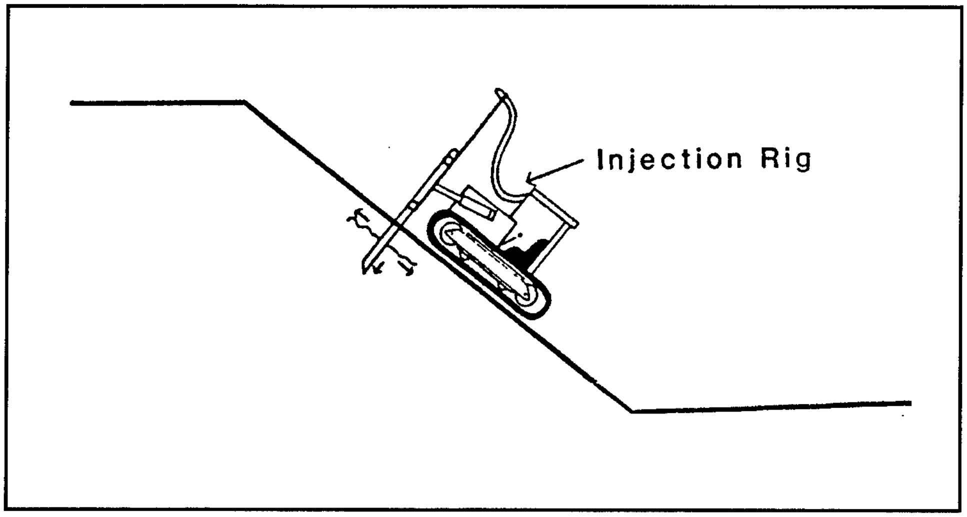


Figure 43. Lime-fly ash injection vehicle on levee slope (adapted from Transportation Research Board, Blacklock and Wright 1986)

the levee rather than parallel to avoid loosening the soil on the slope (Blacklock and Wright 1986; USAED, Kansas City 1989; Baez 1988; Baez, Borden, and Henry 1991; Pengelly and Holloway 1993; Holloway 1994).

The USAED, Kansas City, constructed three test sections using lime-fly ash injection in 1984 on Unit L-246 of the Missouri River Levee System near Brunswick, MO. The entire lengths of both reaches of the right bank of the Chariton River Levee (14,950 linear ft) were lime-fly ash injected in 1987 to 1988 (Pengelly and Holloway 1993, Holloway 1994). The USAED, Memphis and St. Louis, have done limited work with lime-fly ash injection of levees.

The Chariton River Levee is the largest lime-fly ash injection project in the USACE. As stated previously, although shrinkage cracks have occurred in the lime-fly ash injected slopes, no slope failures have occurred. In situ hand vane shear tests conducted on this project indicated an increase in strength within 2 to 3 in. of the lime-fly ash seam 6 months after injection with a calculated overall increase in the strength of the soil of 15 percent. At the same time, laboratory unconfined compression tests on undisturbed samples indicated an overall increase in the strength of the soil of 30 percent (Baez 1988; Baez, Borden, and Henry 1991). Continued monitoring of sites is needed to determine if lime-fly ash injection will protect against slope failure over the long term. Long-term monitoring should include weather data, observation of shrinkage cracks, and test pits in both untreated and lime-fly ash injected areas (Baez, Borden, and Henry 1991). In situ testing, such as cone penetration tests (Figure 45), is particularly well-suited to determining changes in properties of the lime-fly ash injected levee soil with time (U.S. Army Engineer District, Kansas City 1989). The complex structure of the lime-fly ash injected soil may not be conducive to sampling and laboratory testing (Mitchell and Klainer 1987). It is important to realize that properties of the levee (unsaturated soil within the zone of active

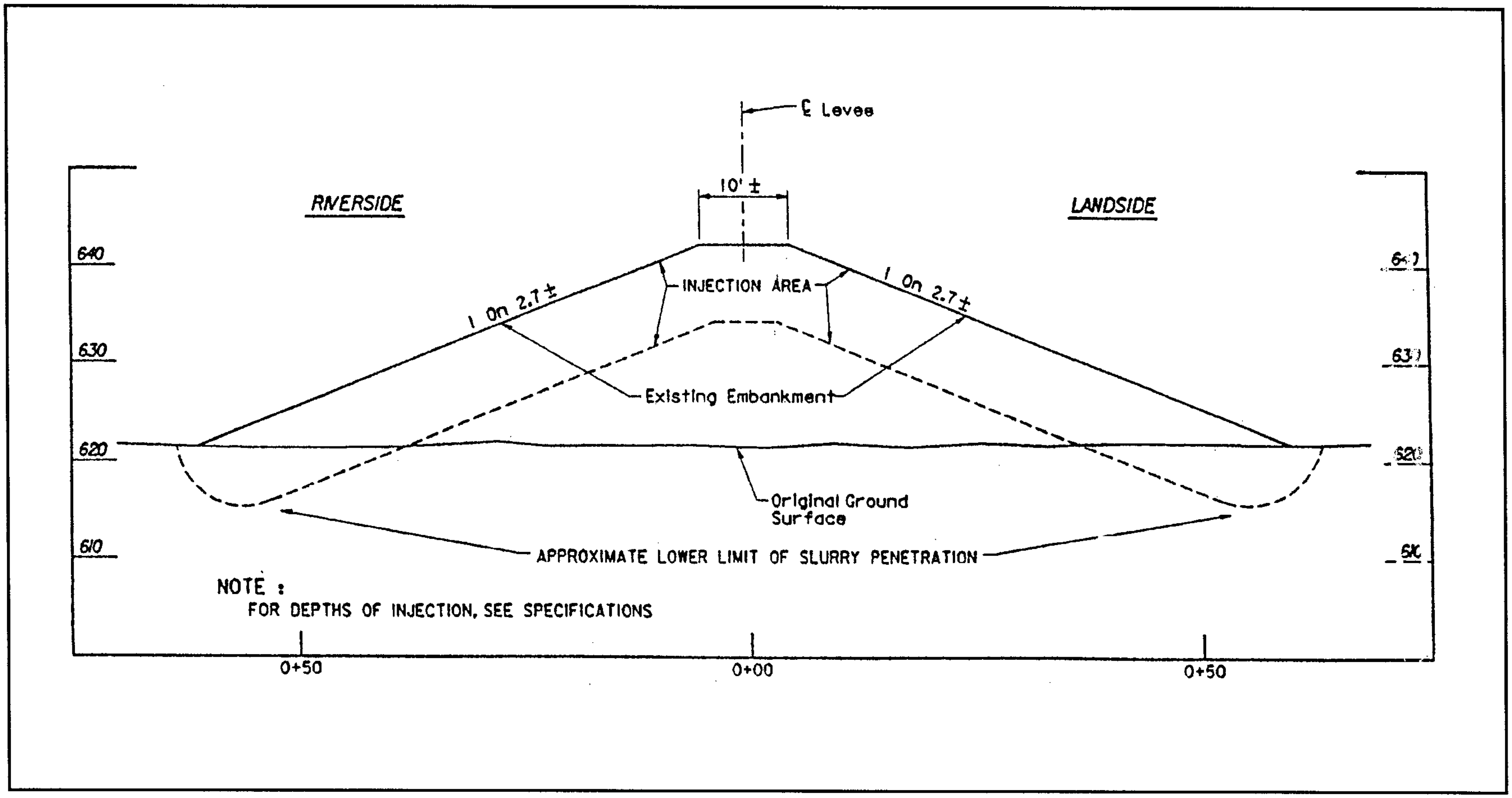


Figure 44. Plan view of levee showing lime-fly ash slurry injection section (after Pengelly and Holloway 1993)

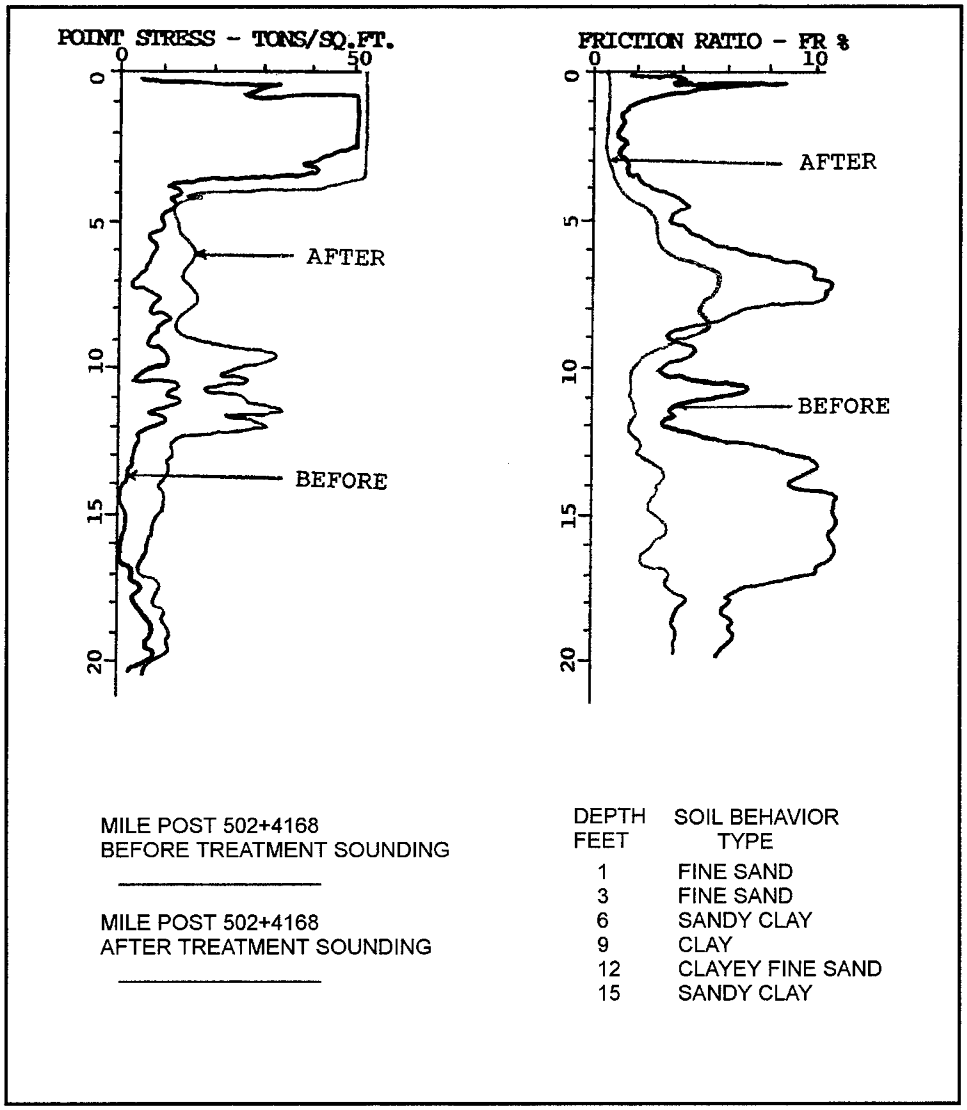


Figure 45. Electric cone penetrometer test results before (May 1987) and after (April 1988) lime-fly ash slurry injection of Santa Fe Railroad embankment section (after GKN Hayward Baker, Woodbine Division) (Undated product literature)

moisture fluctuation) are a function of the weather, particularly extreme events such as droughts and/or high-intensity rainfall, as well as time.

Advantages and disadvantages of lime-fly ash injection are given in Table 5. The advantages of lime-fly ash injection are the relatively low cost, rapid installation, ability to work on wet soft clays, and no excavation required. The disadvantage is that the long-term performance needs to be documented (Baez, Borden, and Henry 1991).

The potential improvement lime-fly ash injection will produce in a soil can be estimated by laboratory tests. Glaze stabilized (coated) compression tests and

split-seam compression tests as shown in Figures 46a and b can be run on undisturbed or remolded samples to evaluate the benefits of lime-fly ash slurry to increase tension, compression, and shear reinforcing strength, respectively. The procedure for conducting the tests is given by Boynton and Blacklock (1985). For remolded samples, the method of compaction used in the laboratory should simulate that used in the field (i.e., kneading compaction in laboratory versus sheepsfoot roller in field). Soil specimens are glaze coated with a lime-fly ash slurry or split, glazed and put together, cured at room temperature for 28 days, and tested in unconfined compression. Untreated control samples are prepared for each experiment. Increase in strength for the glazed soil specimens over untreated control specimens is indicative of favorable response to lime-fly ash injection (Blacklock 1982, Baez 1988).

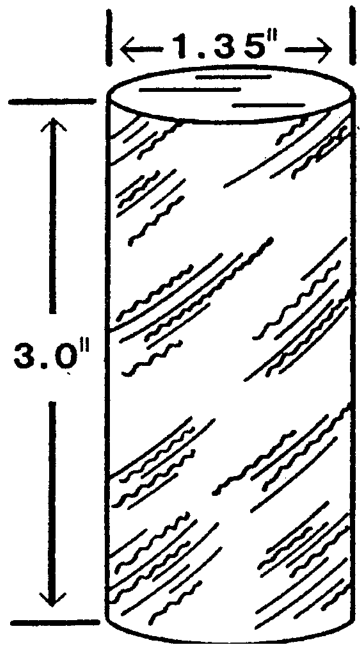
The design of a lime-fly ash injection project on a levee would involve determining the geometry (depth, length, and width) to be injected; the injection pattern to be used (primary, secondary, sequence, etc.); the characteristics of the slurry (pressure, proportions of lime, fly ash, water, and additives such as accelerators and retarders); verification techniques to be employed (test pits, in situ hand vane shear tests, laboratory tests, etc.); and a long-term monitoring plan (weather data, observation of shrinkage cracks and test pits, and in situ testing, such as cone penetration tests in both untreated and lime-fly ash injected areas).

In summary, lime-fly ash injection could be used to repair a failed slope or prevent slope failures from occurring. It is applicable for plastic clays with desiccation cracking. Advantages of lime-fly ash injection are the relatively low cost, rapid installation, ability to work on wet soft clays, and no excavation required. The disadvantage is that long-term performance monitoring (weather data, observation of shrinkage cracks and test pits, and in situ testing, such as cone penetration tests in both untreated and lime-fly ash injected areas) needs to be documented.

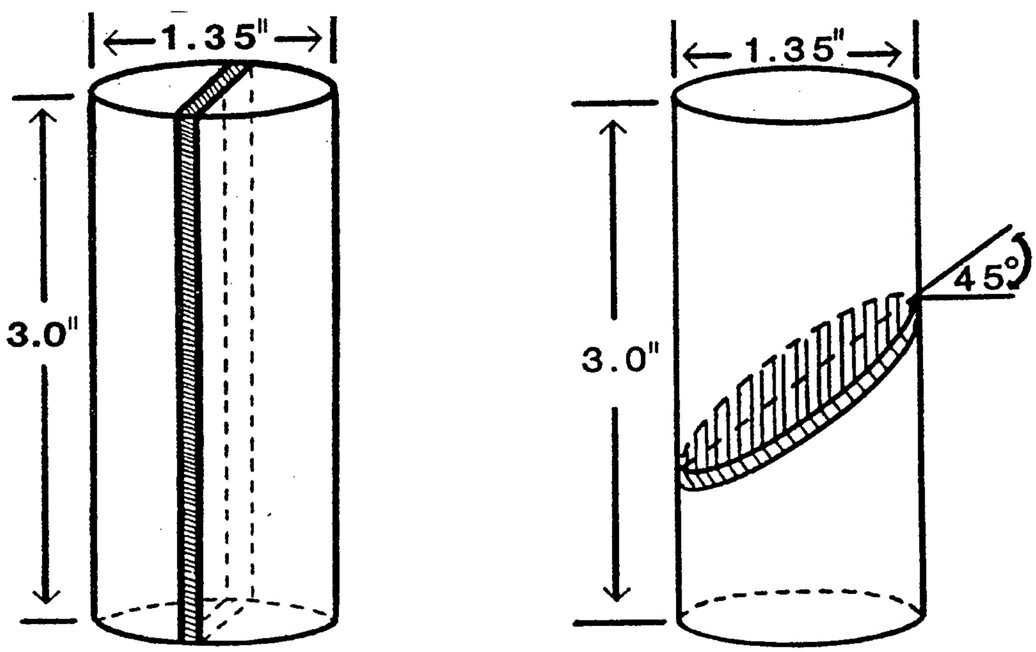
Anchored Geosynthetic System

As previously stated, when the mechanism of slope instability does not involve shrinkage cracks, soil nailing, or an anchored geosynthetic system, also called anchored spider netting, as shown in Figures 47 and 48, may work. Although an anchored geosynthetic system may be used in silts and low-plasticity clays (without shrinkage cracks), anchored geosynthetic systems are best suited to stabilize sandy slopes which have little resistance to sliding in the absence of confining stress near the surface and experience shallow (to a depth of about 10 ft) failure surfaces (Hryciw and Haji-Ahmad 1992, Gray and Sotir 1996).

The geosynthetic material, usually geotextile, geogrid, or geonet, is placed on the unstable or questionable slope and anchored to it with steel rods which extend below the critical failure surface. During installation, the rods are driven to within 75 to 90 percent of their design depth and the anchor lock-off system,



(a) Lime-fly ash glaze stabilized compression test specimen



(b) Split seam lime-fly ash glaze stabilized compression test specimen

Figure 46. Glaze stabilization compression tests and split-seam compression tests to evaluate the potential for lime-fly ash slurry to increase strength of soil (adapted from Transportation Research Board, Blacklock and Wright 1986)

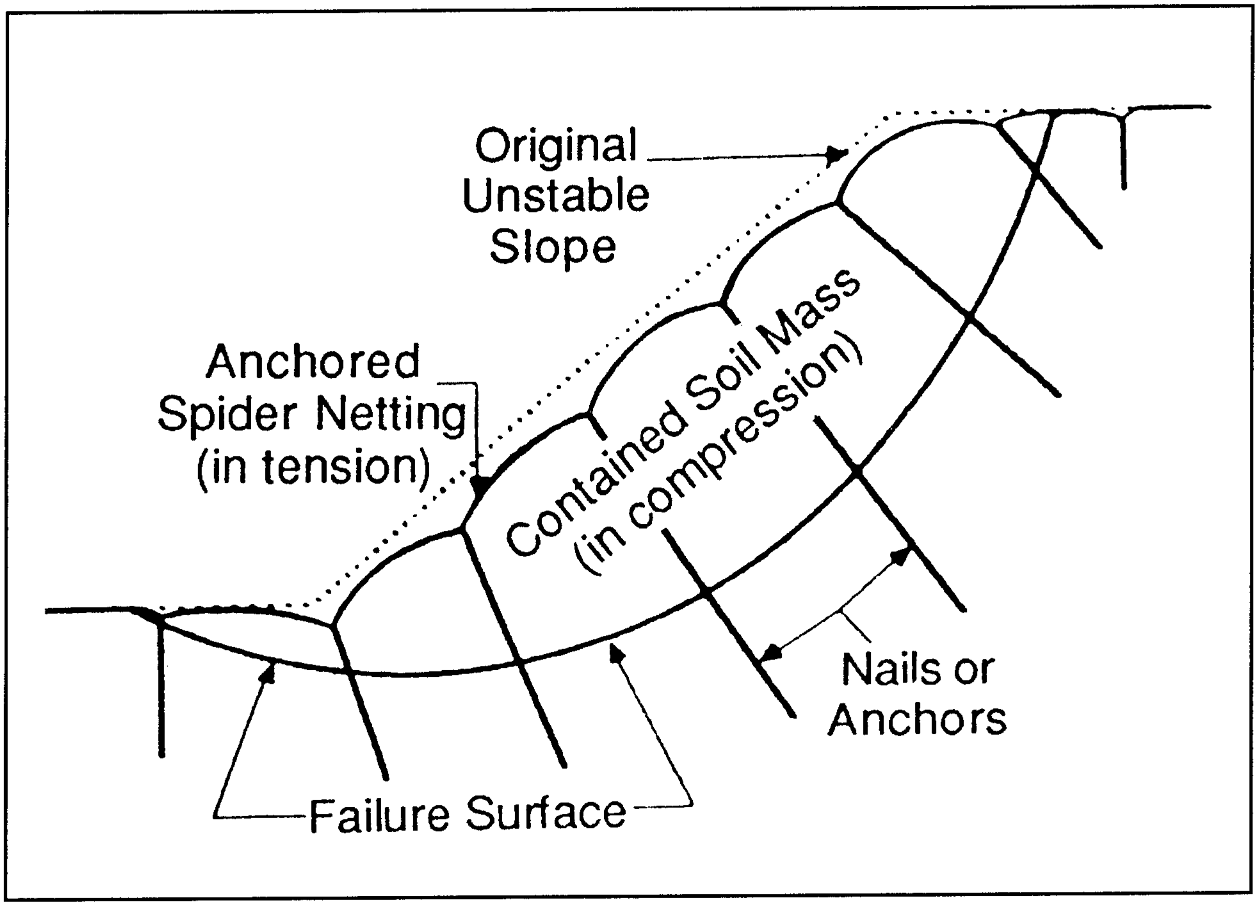


Figure 47. Conceptualized view of anchored geosynthetic system to stabilize a slope (courtesy of Transportation Research Board, Holtz and Schuster 1996)

shown in Figure 49, is fixed. Continued driving tensions the netting as it is pulled along with the anchor. Anchor rods are typically rebars (reinforcing bars) whose ribbed surface gives greater pullout resistance than smooth bars (Irsyam and Hryciw 1991). The geosynthetic system is porous to the flow of water preventing pore water pressure from building up beneath the fabric. If the soil has sufficient cohesion, geogrids or geonets can be used alone and vegetation may be established in the openings. The geosynthetic system should be fine enough to function as a filter and prevent soil from washing through the openings. If this is not the case, the system can be underlain by a geotextile which acts as a filter. Ultraviolet inhibitors may be included during the manufacture of the geosynthetic system, if needed. Geogrids are constructed of polyethylene which are not susceptible to ultraviolet deterioration (Koerner 1984; Koerner 1985; Koerner and Robins 1986; Koerner 1994; Ghiassian, Gray, and Hryciw 1997b).

While soil nailing and reinforced soil slopes, as previously discussed, are passive systems that rely on soil strains to mobilize pullout, bending and shear resistance of the reinforcement, anchors in the anchored geosynthetic system are actively tensioned during installation, and the soil is placed above the anchor in

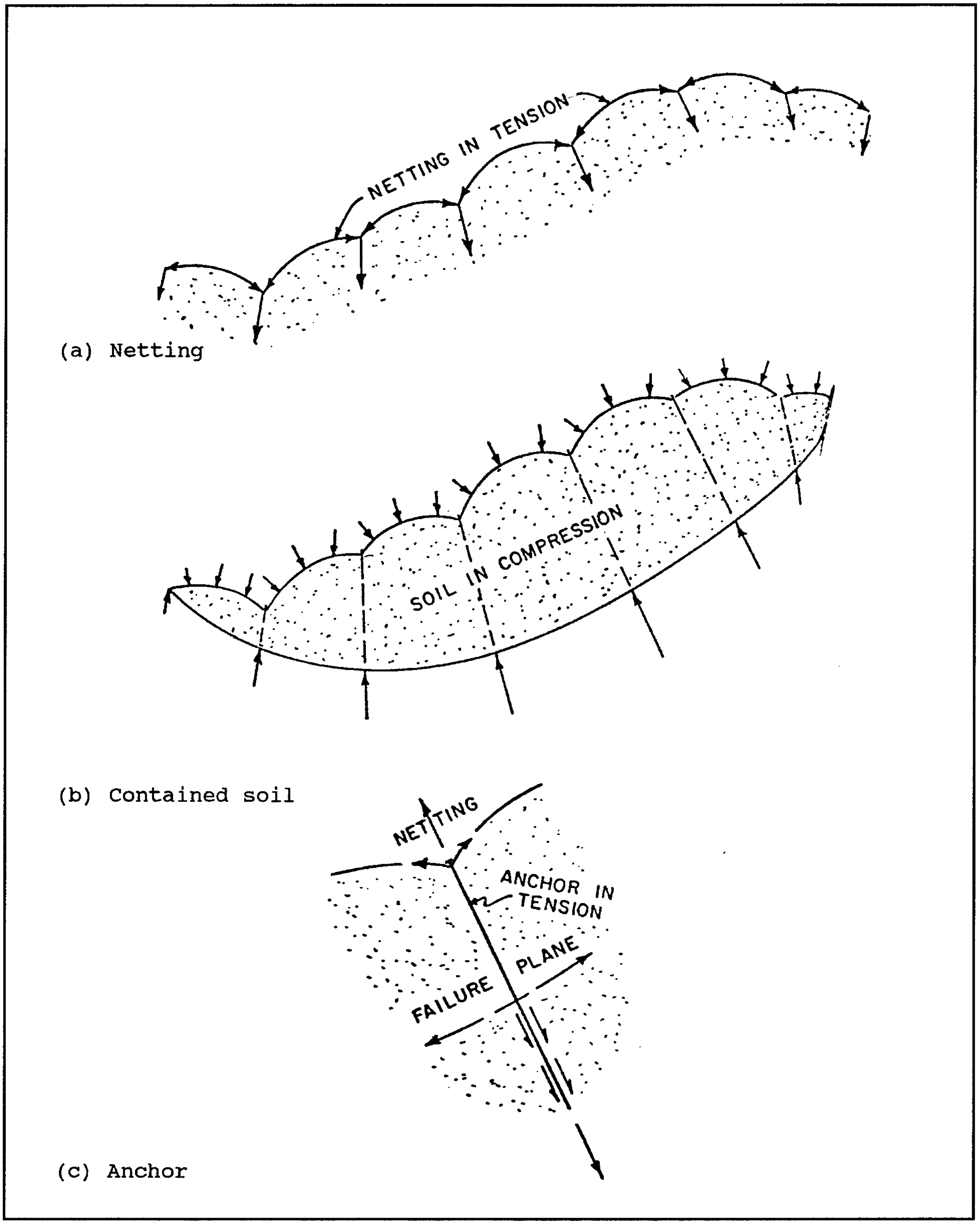


Figure 48. Schematic free body diagrams of an anchored geosynthetic system (courtesy of Envo Publishing Company, Koerner 1984)

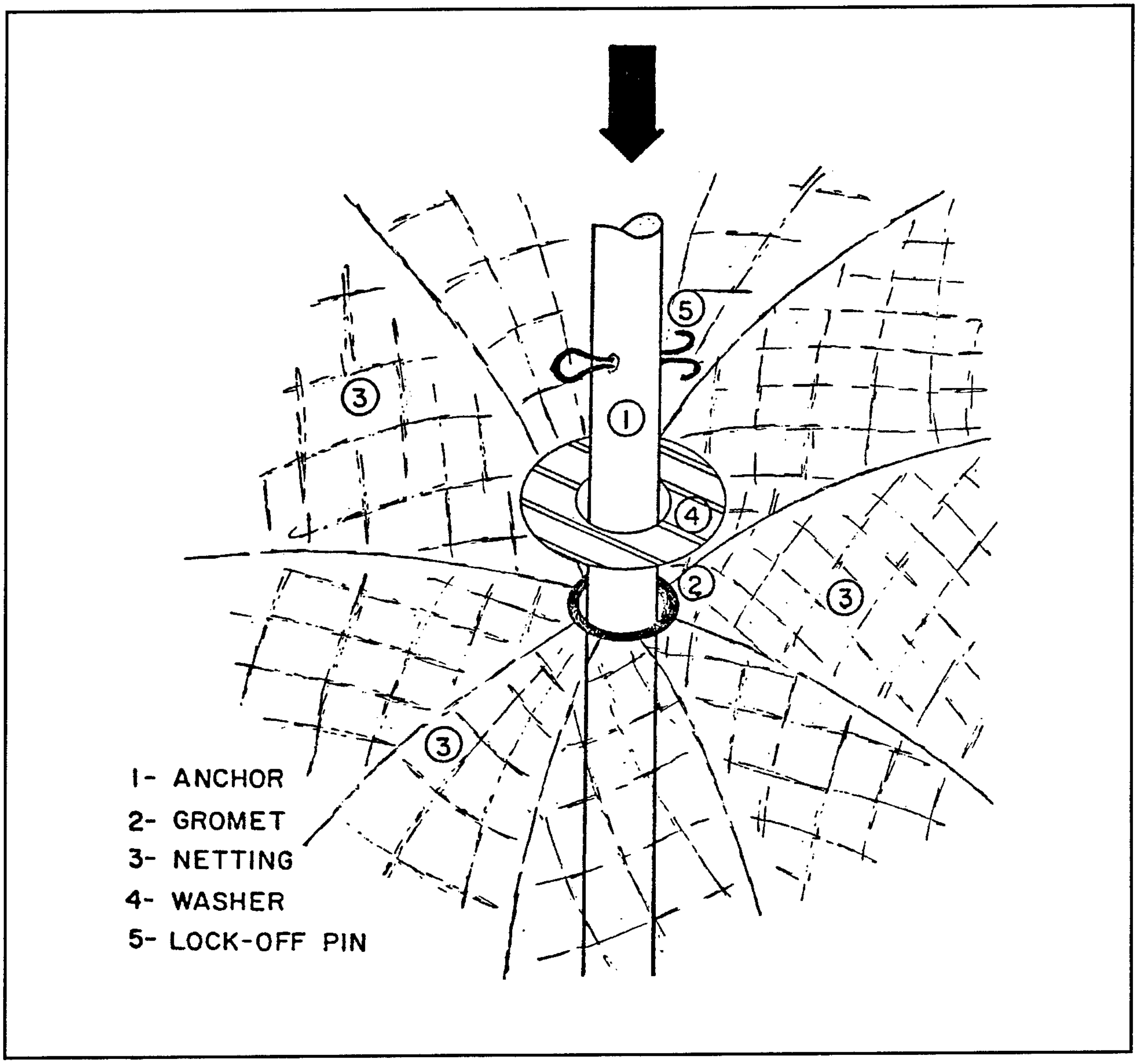


Figure 49. Sketch of anchor lock-off assembly at locations where anchor passes through gromet in netting of anchored geosynthetic system (courtesy of Envo Publishing Company, Koerner 1984)

compression. Therefore, the increase in stability of the slope does not require soil movement to mobilize soil-anchor interaction but rather the increased normal stress on the potential failure surface from the tensioned netting increases the stability of the slope, particularly for sandy soils (Gray, Hryciw, and Ghiassian 1996).

The increased normal stress exerted by the tensioned netting, which is directly proportional to the tension in the fabric and inversely proportional to the radius of curvature of the surface, is not large in magnitude. Significant curvature of the soil-fabric interface develops to a distance of about 1.5 ft from

the anchor. Beyond this area at shallow depth, the soil between anchor points will be subject to little confining stress increase. For typical anchor spacings of 5 to 8 ft, the pressure increase on a potential failure surface at a depth of 0.75 times the anchor spacing will be less than 10 percent. Consolidation of the soil and stress relaxation in the geosynthetic may require anchor redriving following initial installation (Vitton and Hryciw 1991; Hryciw 1991; Gray and Sotir 1996; Ghiassian, Gray, and Hryciw 1997b).

Recent research shows that the effectiveness of an anchored geosynthetic system is related to the geometric arrangement, spacing, and inclination of anchors; visco-elastic properties of the geosynthetic system; soil-geosynthetic interaction friction and stretched shape of the geosynthetic system on the ground. When properly anchored, the geosynthetic system should provide shallow mass stability and erosion resistance to the slope. Various anchor point arrays for an anchored geosynthetic system are shown in Figure 50. Erosion resistance is greatest (narrow erosion channel and most tortuous path) for the triangular anchor pattern with decreased row spacing, as shown in Figure 50c. The efficiency of stress transfer can be increased by digging conical depressions in the ground at anchor point locations (would be labor intensive) or using rigid horizontal bars attached to the fabric and connected to the anchor rods placed in parallel furrowed rows running along the slope's contours (Ghiassian, Gray, and Hryciw 1997b).

For slopes subjected to seepage, as would be the case for sandy slopes, insertion of relatively short drains into the slope at anchor points can significantly improve the stability of the slope and piping resistance of the soil by preventing seepage water from exiting the slope at points beneath the anchored geosynthetic system (Ghiassian, Gray, and Hryciw 1997a).

The fabric should have a wide-width tensile strength of 200 lb/in. with local reinforcement at anchor points equivalent to 500 lb/in. wide-width tensile strength for 6 in. around the anchor (Koerner 1984, 1994). To avoid excessive stress relaxation and creep in the geosynthetic system at high stresses, the fabric should be loaded to less than 60 percent of its tensile strength and working stresses should be limited to less than 40 percent of the peak strength (Gray, Hryciw, and Ghiassian 1996; Gray and Sotir 1996; Hryciw 1990; Ghiassian, Gray, and Hryciw 1997b).

Theoretically, for maximum efficiency, the anchors should be installed at an orientation which maximizes the resistance. For a normally consolidated sand, with anchor points installed in a square pattern, the optimum anchor orientation, which achieves a desired factor of safety against slope instability for a minimum anchor length to spacing ratio, is (Hryciw 1991):

$$\theta_{opt} = 47.5^{\circ} - 0.7\,\beta - 9\,K + 8\,FSR$$

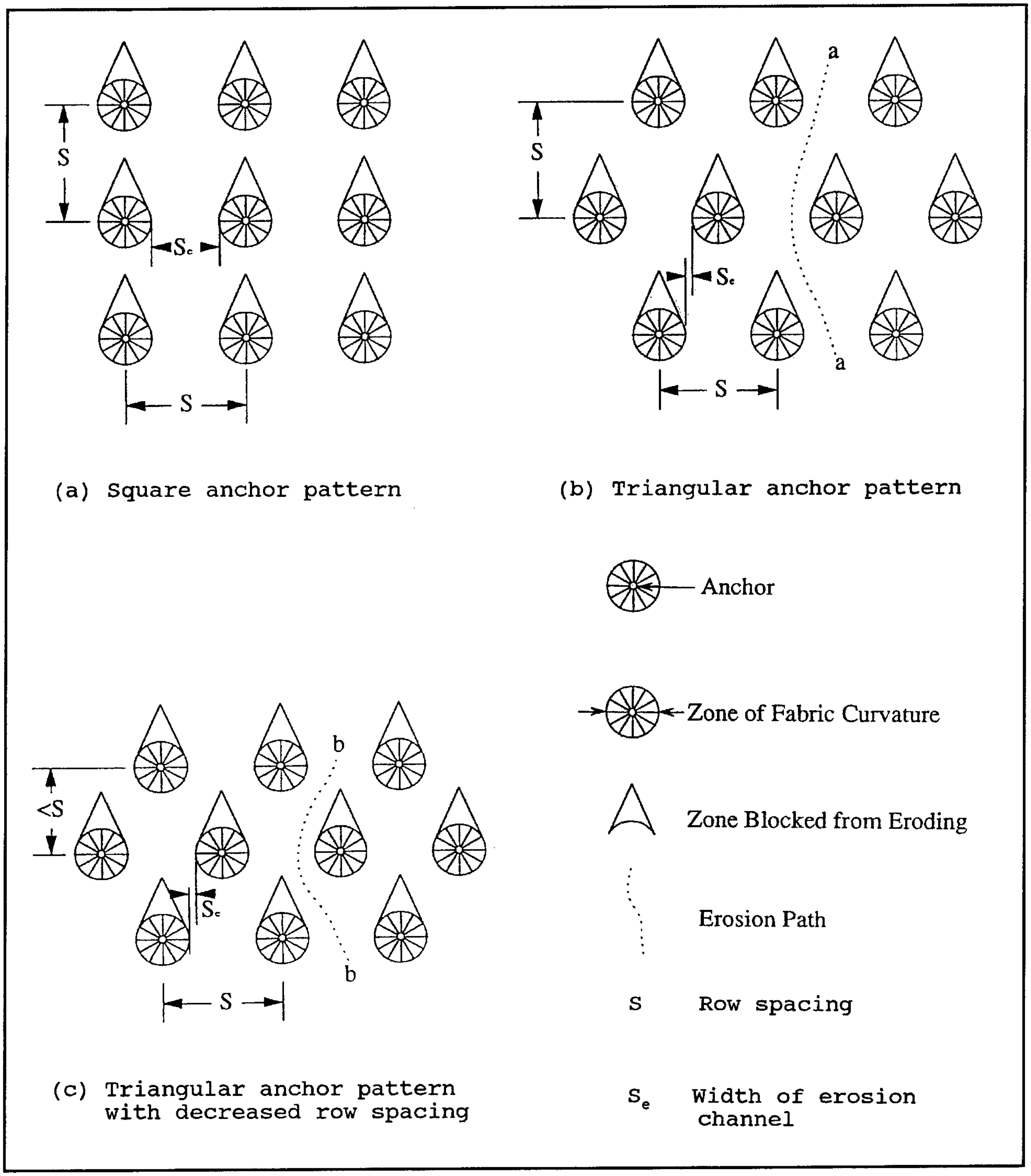


Figure 50. Anchor arrays for anchored geosynthetic system (after Hryciw 1990)

where

θ_{opt} = Optimum anchor orientation angle measured from a perpendicular to the slope at which the factor of safety is maximized (Figure 51)

β = Slope angle

K = Coefficient of lateral earth pressure at rest

FSR = Ratio of factor of safety with anchors to factor of safety prior to installing anchors

It may not be practical or convenient to install anchors at an angle other than perpendicular to the slope using hand-held vibratory percussion tools. The anchors should not be installed with inclinations above the horizontal (Figure 51) because handling of the installation equipment would be difficult and local soil ravelling could occur (Hryciw and Haji-Ahmad 1992).

Advantages and disadvantages of anchored geosynthetic systems are given in Table 5. The advantages of anchored geosynthetic systems are the relatively low cost, use of light equipment, rapid construction, no requirement for skilled labor or special equipment, and adaptability for environmentally sensitive areas because anchored geosynthetic systems are physically intrusive, resistant to erosion, and promote establishment of vegetation (Ghiassian, Gray, and Hryciw 1997b). The disadvantage is that anchored geosynthetic systems may restrict activities on the levee such as mowing and grazing livestock until vegetation is well established.

Design of anchored geosynthetic systems includes determination of the optimum anchor orientation angle, length of the anchors, spacing and pattern of the anchors. Additional factors to consider are fabric tensile strength and filtration, stress increase distribution, erosion control, and installation details (Hryciw and Haji-Ahmad 1992). Methods for designing anchored geosynthetic systems are given by Koerner (1984, 1985, 1994), and Koerner and Robins (1986), and Hryciw and co-workers (Vitton and Hryciw 1991; Irsyam and Hryciw 1991; Hryciw 1991; Hryciw and Haji-Ahmad 1992; Gray, Hryciw, and Ghiassian 1996; Ghiassian, Gray, and Hryciw 1997a,b).

In summary, anchored geosynthetic systems are best suited to stabilize sandy slopes which have little resistance to sliding in the absence of confining stress near the surface and experience shallow (to a depth of about 10 ft) failure surfaces. Use of short drains at anchor points can significantly improve the stability of the slope and piping resistance of the soil. Advantages of anchored geosynthetic systems are a relatively low cost, use of light equipment, rapid construction, no requirement for skilled labor or special equipment, and well-suited for environmentally sensitive areas because anchored geosynthetic systems are physically intrusive, erosion resistant, and they promote establishment of vegetation. The disadvantage is that anchored geosynthetic systems may

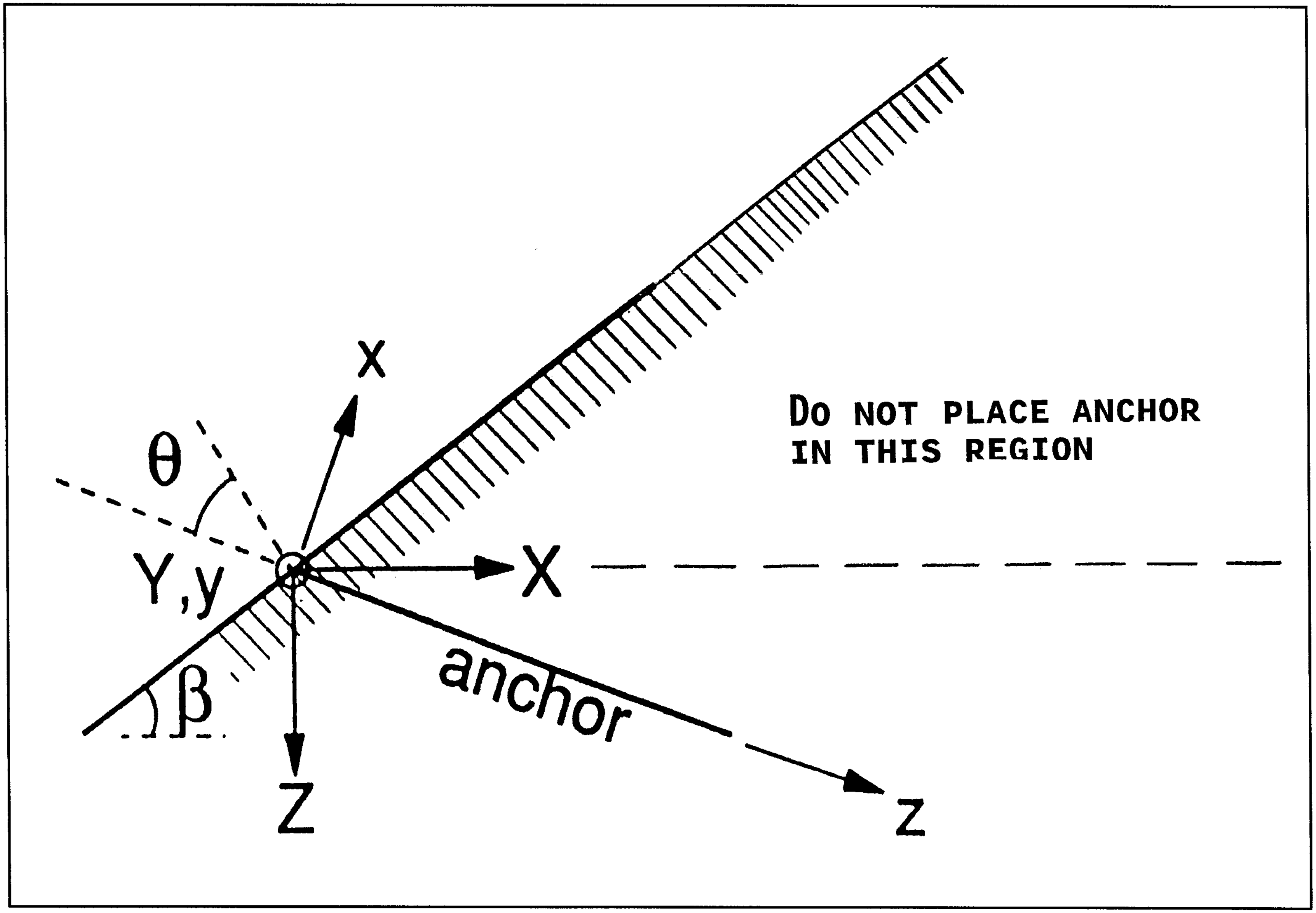


Figure 51. Coordinate system of anchored geosynthetic system showing region to place anchor

restrict activities on the levee such as mowing and grazing livestock until vegetation is well established.

Overview of Innovative Methods for Slope Instability

Overview and applications of innovative methods for slope instability are given in Tables 5 and 8, respectively. Reinforced soil slopes can be used with all soils (silts, sands, and clays with or without desiccation cracking) for all purposes (prevent creep, prevent failure, or stabilize a failed slope) and meet the requirements of other conditions (depth of failure surface, stable during excavation, and competent underlying strata) to rebuild levees to any slope angle up to 90 deg. Reinforced soil slopes, therefore, are particularly useful when right-of-way is limited, such as for urban levees. Soil nailing can be used with all soils, except plastic clays with desiccation cracking or soft clays with significant creep, to prevent failure of levees. Soil nailing may not be applicable where underground utilities are present. Pin piles can be used with all soils, where there is a competent underlying strata, to prevent failure. Since some movement is required with pin piles to mobilize support, they would not be applicable in the vicinity of structures where such movement was unacceptable. Stone-fill trenches can be used to stabilize failed slopes with all soils that remain stable during excavation to a depth below the failure surface. Randomly distributed synthetic fibers in their present configuration, i.e., short and smooth, are not recommended for slope stabilization in clay subjected to desiccation cracking. Restraint structures can be used in cohesive soils, where there is a competent underlying strata (lower part of the levee or foundation) within about 20 ft or less, to stabilize failed slopes. Construction cost increases rapidly with increased height of wall. Geosynthetic drainage systems can be used in clays with desiccation cracking as a temporary measure to prevent failure. Maintenance is required to prevent possible blockage of outlets by silt accumulation and/or grass root intrusion. Lime-fly ash injection can be used in clays with desiccation cracking to prevent creep, prevent failure, or stabilize a failed slope. Long-term performance needs to be documented for lime-fly ash injection. Anchored geosynthetic systems can be used with sandy slopes to prevent shallow ($\leq$ 10 ft) surface failure. This method restricts activities (mowing, livestock) until vegetation is well established.

Table 8
Applications of Innovative Rehabilitative Methods for Slope Instability

Rehabilitative Method	Soil Conditions and Purpose				
	Soil Conditions		Purpose		
	Type	Desiccation Cracking	Prevent Creep	Prevent Failure	Stabilize Failed Slope
Reinforced soil slope	All	Yes	Yes	NA	Yes
Soil nailing	All, with exception of plastic clays with desiccation cracking	No	Limited	Yes	No
Pin Piles	Most	Yes	Yes	Yes	No
Stone-fill trenches	Most	Yes	Yes	NA	Yes
Randomly distributed synthetic fibers	Not recommended with short smooth fibers	Yes	Yes	NA	Yes
Restraint structure	Cohesive	Yes	No	Yes	Yes
Geosynthetic drainage system	Plastic clays with desiccation cracking	Yes	No	Yes	No
Lime-fly ash injection	Plastic clays with desiccation cracking	Yes	Yes	Yes	Yes
Anchored geosynthetic system	Best suited for sandy slopes with shallow (≤ 10 ft) failure surface	No	No	Yes	No

Other Conditions				
Rehabilitative Method	Depth of Failure Surface	Stable During Excavation	Competent Underlying Strata	Special Conditions
Reinforced soil slope	NA	NA	NA	Slope can be rebuilt to any angle up to 90 deg
Soil nailing	NA	NA	NA	May not be feasible with underground utilities
Pin piles	NA	NA	Yes	Some movement required to mobilize support
Stone-fill trenches	NA	Yes	NA	Stone must be available for backfilling
Randomly distributed synthetic fibers	NA	NA	NA	Short smooth fibers do not deter cracking when subjected to wet/dry cycles
Restraint structure	Shallow	NA	≤ 20 ft	Cost increases with increased height of wall
Geosynthetic drainage system	≤ depth of desiccation cracking	Yes	NA	Temporary measure and maintenance required

(Continued)

Table 8 (Concluded)

Other Conditions				
Rehabilitative Method	**Depth of Failure Surface**	**Stable During Excavation**	**Competent Underlying Strata**	**Special Conditions**
Lime-fly ash injection	NA	NA	NA	Long-term performance needs to be documented
Anchored geosynthetic system	$\leq$ 10 ft	NA	NA	Restricts activities on levee (mowing, livestock) until vegetation is well established

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The Corps of Engineers is responsible for 8,500 miles of levees. Levees are subject to overtopping, current and wave attack, surface erosion due to rainfall, through-seepage, underseepage, and slope instability. Conventional methods of levee rehabilitation are both costly and time consuming. Research is underway to develop new methods of levee rehabilitation. This report covers analytical studies on two-dimensional slope stability analysis of geogrid mechanically stabilized slides and slope remediation using a prefabricated geocomposite drainage system. Results of laboratory studies to determine the effectiveness of short randomly distributed polypropylene fibers in reducing crack development (and shallow surface sliding) in clay slopes are given. Guidelines are presented for applying innovative chemical and physical techniques such as geosynthetic systems, mechanically stabilized soil, soil nailing with geotextiles, gravel trenches, and slide suppressor walls to levee rehabilitation.

14. SUBJECT TERMS

Concrete blocks	Geotextiles	Overtopping
Erosion	Levee rehabilitation	Seepage
Geocells	Lightweight materials	Slope stability
Geogrids	Mechanically stabilized soil	Soil nailing

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